Mitigation of Cracking in Cast-In-Place Reinforced Concrete Box Culverts in Alabama

by

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Abstract

Significant cracking was observed in Alabama Department of Transportation (ALDOT) cast-in-place (CIP) reinforced concrete box culverts in the Anniston East Bypass (AEB) project in Anniston, AL. Numerous wide, transverse cracks were observed inside the culvert barrels, and cracking was also observed in the culvert wingwalls.

Because of the cracking problems in the AEB project, crack condition surveys were done of other CIP reinforced concrete box culverts throughout Alabama to investigate the distress. An instrumentation and testing plan was also developed for a CIP reinforced concrete box culvert under construction in order to evaluate the stress, strain, and temperature development as well as other properties. The amount of temperature and shrinkage reinforcement required to produce acceptable average crack widths in CIP reinforced concrete box culverts was investigated as well.

It was concluded that the transverse cracking was most likely a result of restrained thermal and drying shrinkage deformations in the concrete. Transverse contraction joints in the culvert barrel and vertical wingwall joints were proposed to control the occurrence of these cracks. The amount of longitudinal temperature and shrinkage reinforcement was also recommended to be increased to control the crack widths to 0.012 in. (0.30 mm).

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Chapter 1

Introduction

1.1 Background

1.1.1 Box Culverts

Box culverts are essential structures as they help shield the bases, ditches, and banks of roads from erosion and other detrimental effects by channeling the runoff water into roadside ditches. They allow existing streams to flow under roadways, and keep the road sub-bases dry. Installing culverts at strategically chosen locations allows for control of runoff water flow velocity and for the prevention of roadway flooding by maintaining the design flow capacity of the ditches. All of this helps to achieve the overall goal of reducing road maintenance and upkeep. (Choctawhatchee 2000)

Along with being a hydraulic structure, box culverts must also be designed to withstand various types of loads. They must be able to support lateral loads from earth pressure as well as vertical loads from earth and vehicle pressures. (WisDOT 2011)

While most culverts generally fulfill the same purpose, they can be classified differently. The Alabama Department of Transportation (ALDOT) and Florida Department of transportation (FDOT) classify box culverts with spans less than 20 feet as culverts, and box culverts with spans longer than 20 feet are classified as bridge culverts (ALDOT 2008; FDOT 2011).

Culverts can be made out of different materials. They are typically made of concrete, corrugated metal, or plastic. The Florida Department of Transportation (2011) states that concrete culverts are generally preferred over corrugated metal and plastic when life-cycle costs are considered. They are initially more expensive, but their resistance to environmental conditions, corrosion resistance, hydraulic efficiency, and long service life make them attractive (FDOT 2011).

Reinforced concrete box culverts can be precast or cast-in-place (CIP). Using precast structures allows for concrete structures to be mass produced at a plant and delivered to the site (FDOT 2011; KYTC 2011). It also allows for the reduction of issues associated with construction time, site constraint, traffic management, and stream diversion. However, using precast culverts only allows for certain sizes and skews to be used, as a result of transportation and handling concerns. In addition, the cost of transportation to the job site can be high, and can overcome its advantages. (FDOT 2011) CIP reinforced concrete box culverts are built at the construction site (KYTC 2011). They are typically built when ready-mix concrete can be obtained and when it is desired to keep the number of transverse joints to a minimum. An advantage of using the CIP method is that the culverts can be specifically designed to meet the unique needs of the site. (ConnDOT 2000)

Culvert wingwalls can be described as retaining walls attached to the ends of culverts to retain fill material and to direct flow. CIP wingwalls are preferred, but precast wingwalls can be used in certain cases. (FDOT 2011) Wingwalls are to be used, along with headwalls, as a retaining wall for the roadway embankment (VDOT 2002). They serve the following purposes: to limit seepage, make the culvert ends structurally sound, retain fill material, improve hydraulic features, reduce erosion, and enhance aesthetics. Wingwalls may or may not be attached to the headwall. (KYTC 2011)

Box culverts can be constructed with more than one barrel. Multiple-barrel, CIP culverts are usually built with all barrels having a uniform size. This is so that standard details can be used. (VDOT 2002) However, using multiple-barrel culverts may produce problems. They can generate a build-up of debris over time, which can cause blockage. Erosion problems can also arise due to the presence of multiple barrels. Single barrel openings are preferred unless life-cycle analysis shows that the savings from the costs of construction will be greater than the cost of maintenance. (TDOT 2010)

1.1.2 Culvert Cracking in the Anniston East Bypass (AEB)

Multiple CIP reinforced concrete box culverts were built on the Anniston East Bypass (AEB) in Anniston, Alabama by the Alabama Department of Transportation (ALDOT). Within two years after construction, these culverts showed excessive transverse cracking. Cracks wider than 0.012 in. (0.30 mm) are considered a hazard to the culvert's durability (ACI 224 2001), and many of the observed AEB culvert cracks were wider than 0.04 in. (1 mm). Illustrations of some cracks found in the AEB project are shown in Figures 1-1 and 1-2.

Figure 1-1: Transverse Crack in the AEB Project (Crack Width ≈ 0.08 in. [2 mm])

Figure 1-2: Transverse Base Crack in the AEB Project (Crack Width > 0.06 in. [2 mm])

In a preliminary crack survey of AEB Culvert at 240+37 (Culvert J), performed by ALDOT on March 11, 2009, many wide transverse cracks in the concrete between construction joints were observed. A majority of the cracks were located in the top slab and walls. Some cracks were observed in the walls only. Efflorescence was found around a majority of the cracks indicating that the cracks had not happened recently and that they ran completely through the top slab. Water was leaking through many of the cracks. Longitudinal cracks were also observed at the bottom of the chamfer between the top slab and walls, and in the center of the top slab. The chamfer cracks appeared to run the length of the entire culvert. The longitudinal cracks in the center of the top slab occurred between the french drains within the first 200 feet (60.1 m) of the south end of the culvert. Cracks in the headwall and the wingwalls of the culvert were also observed.

Follow up crack surveys were performed by an Auburn University research team and similar observations were made. Most of the transverse cracks found were in the walls. Few transverse base cracks were found, but they tended to be very wide (the widest crack found was greater than 0.10 in. [2.5 mm]). Tight longitudinal chamfer cracks (less than 0.01 in. [0.3 mm]) were observed at the ends of the culvert too. Vertical cracks in the wingwalls and diagonal cracks in the headwall were observed. Efflorescence and very limited signs of corrosion were found at some of the cracks.

AEB Culvert at 257+69 (Culvert I) and AEB Culvert at 149+60 (Culvert C) were also surveyed by the Auburn University research team. AEB Culvert at 149+60 was surveyed because it had the same barrel size and construction joint detail as the AEB Culvert at 240+37. Wall and ceiling cracks in excess of 0.012 in. (0.30 mm) were found, as well as openings in the construction joints.

The transverse construction joints in culverts AEB Culvert at 240+37 and AEB Culvert at 149+60 were specified to be ¾” Vee Joints with the reinforcement running continuously through them. See Figure 1-3 for an illustration of the vee joint. This joint was meant to keep any movement other than cracking from occurring at the joint.

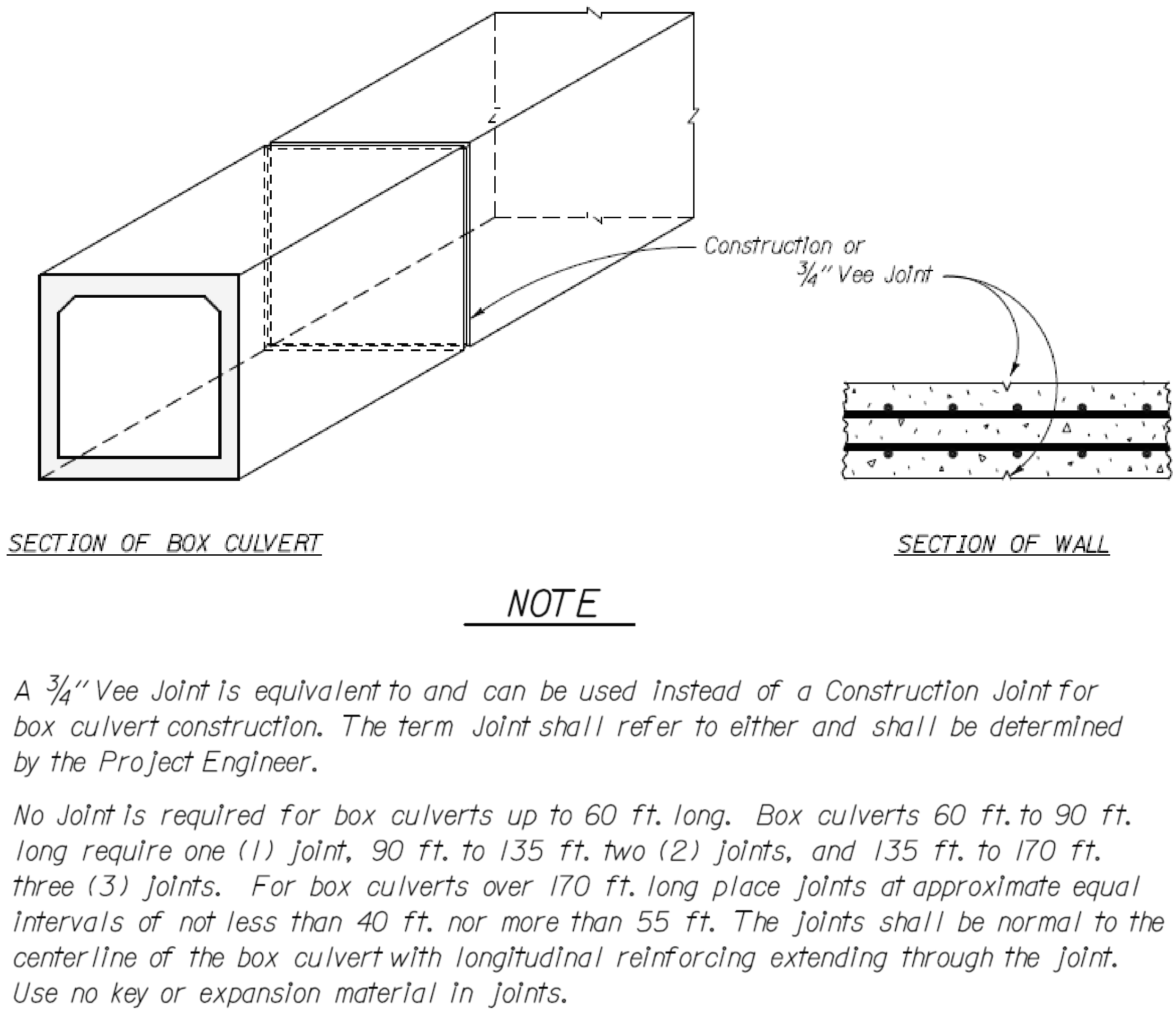


Figure 1-3: Vee Joint Detail (ALDOT 2010)

1.2 Objectives

This research project is sponsored by the Alabama Department of Transportation, and it is in response to the discovery of severe cracking in the Anniston East Bypass (AEB) culverts. The objectives of the researched performed are

- Determine the extent of cracking in other cast-in-place reinforced concrete box culverts in Alabama,
- Develop an instrumentation plan to assess the development of stresses in newly constructed culverts,
- Determine the mechanisms that cause significant cracking in cast-in-place reinforced concrete box culverts, and
- Develop recommendations for ALDOT to mitigate the occurrence of cracking in cast-in-place reinforced concrete box culverts.

1.3 Research Approach

To achieve the objectives stated in Section 1.2, culvert crack condition surveys were conducted of existing CIP reinforced concrete box culverts throughout Alabama. The culvert crack condition surveys consisted of documenting the width and location of all of the transverse cracks observed. The locations of any other signs of distress were also documented.

The instrumentation and testing plan for a future CIP reinforced concrete box culvert under construction was developed. This plan included monitoring the concrete stress, strain, and temperature development in the culvert walls. A system to document the crack width progression of transverse cracks in the culvert was also outlined. The tests outlined in the instrumentation and testing plan included assessments of creep, drying shrinkage, tensile strength development, modulus of elasticity development, compressive strength development, maturity, setting time, and early-age restrained stress development. Quality control testing was also outlined.

The amount of temperature and shrinkage reinforcement required in a CIP box culvert to keep the average crack width at or below 0.012 in. (0.30 mm) was examined. An analysis procedure developed by Gilbert (1992), which was modified to include joint movement and thermal shrinkage, was used to determine the amount of temperature and shrinkage steel necessary. The analysis results were compared to other temperature and shrinkage recommendations from various sources.

The culvert construction and design practices of the American Association of State Highway and Transportation Officials (AASHTO) and states that have similar climates to Alabama were researched in order to develop cracking mitigation methods. Information from this research was used to develop a design for contraction and expansion joints to be used in the culvert barrels and in the wingwalls.

1.4 Definitions and Terminology

1.4.1 Culvert Terminology

When the terms ceiling, interior wall, exterior wall, base, wingwall, footing/foundation, and culvert barrel are used in this report, they refer to the elements shown in Figure 1-4 below.

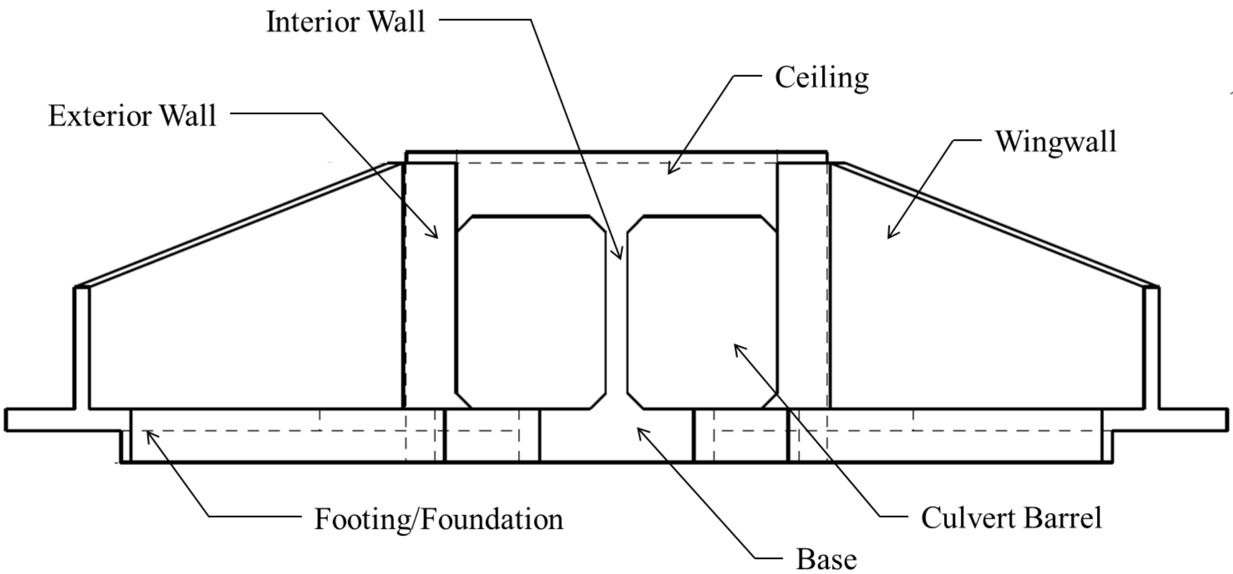


Figure 1-4: Culvert Components (ALDOT 2011)

1.4.2 Definitions

The following terms used in this thesis are defined as follows by the American Concrete Institute (2010):

- **Cast-in-Place Concrete** — concrete that is deposited and allowed to harden in the place where it is required to be in the completed structure, as opposed to precast concrete.
- **Construction Joint** — the surface where two successive placements of concrete meet, across which it may be desirable to achieve bond and through which reinforcement may be continuous.
- **Contraction Joint** — a formed, sawed, or tooled groove in a concrete structure to create a weakened plane to regulate the location of cracking resulting from the dimensional change of different parts of the structure.

- **Diagonal Crack** — in a flexural member, an inclined crack caused by shear stress, usually at about 45 degrees to the axis; or a crack in a slab, not parallel to either the lateral or longitudinal directions.
- **Dowel** — (1) a steel pin, commonly a plain or coated round steel bar that extends into adjoining portions of a concrete construction, as at an expansion or contraction joint in a pavement slab, so as to transfer shear loads; or (2) a deformed reinforcing bar intended to transmit tension, compression, or shear through a construction joint.
- **Hairline Crack** — a concrete surface crack with a width so small as to be barely perceptible.
- **Joint Filler** — compressible material used to fill a joint to prevent the infiltration of debris and provide support for sealants applied to the exposed surface.
- **Joint Sealant** — compressible material used to exclude water and solid foreign materials from joints.
- **Longitudinal Crack** — a crack that develops parallel to the length of a member.
- **Longitudinal Joint** — a joint parallel to the length of a structure or pavement.
- **Precast Concrete** — concrete cast elsewhere than its final position.
- **Plastic-Shrinkage Crack** — surface crack that occurs in concrete prior to initial set.
- **Transverse Crack** — a crack that crosses the longer dimension of the member.
- **Transverse Joint** — a joint normal to the longitudinal dimension of a structural element, assembly of elements, slab, or structure.
- **Waterstop** — a thin sheet of metal, rubber, plastic, or other material inserted across a joint to obstruct the seepage of water through the joint.

An illustration of longitudinal and transverse cracks in a box culvert, to go along with the terminology above, is shown in Figure 1-5.

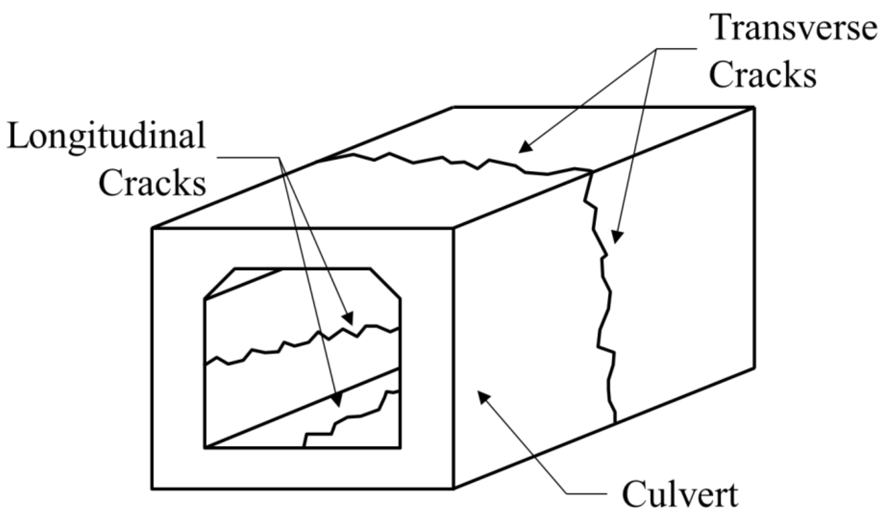


Figure 1-5: Longitudinal and Transverse Crack Illustration in a Box Culvert

1.4.3 Joint Terminology

The ALDOT term “vee joint” is used to refer to construction joints that are not designed to allow movement. Figure 1-6 is an illustration of a vee joint. The vee joints in ALDOT culverts may not always have the ¾ in. indention shown in Figure 1-3, but they have continuous reinforcement.

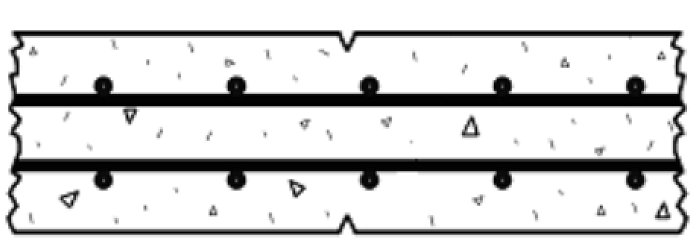


Figure 1-6: ALDOT Vee Joint (ALDOT 2010)

Joints where movement is allowed are referred to as contraction or expansion joints. These joints may also serve the purpose of construction joints. A contraction joint used in the AEB project is shown in Figure 1-7.

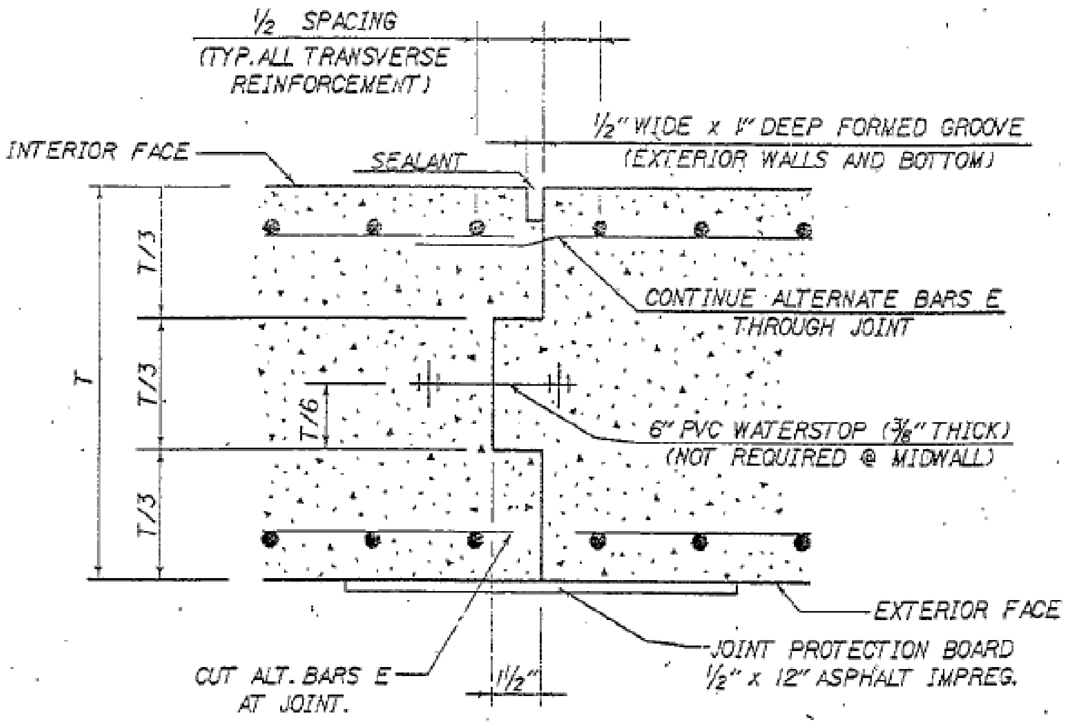


Figure 1-7: Contraction Joint from the AEB Project (ALDOT 2001)

1.5 Thesis Outline

A review of literature on relevant topics is included in Chapter 2 of this thesis. Topics reviewed include thermal stresses, plastic shrinkage, autogenous and chemical shrinkage, drying shrinkage, corrosion, inadequate curing, crack control, crack repair, American Association of State Highway and Transportation Officials (AASHTO) construction and design practices, and culvert construction and design practices of states that are members of the Southeastern Association of State Highway and Transportation Officials (SASHTO).

A summary of the CIP reinforced concrete box culverts visited in Alabama is included in Chapter 3. The procedure used during the crack surveys, the sites visited, the data collected, and conclusions are covered in this chapter.

In Chapter 4, the amount of temperature and shrinkage reinforcement required to keep the average crack widths at or below the ACI 224 (2001) limit of 0.012 in. (0.30 mm) in a CIP reinforced concrete box culvert is investigated. The analysis procedure that is used to calculate average crack widths is presented, along with recommendations for the ratio of temperature and shrinkage reinforcement that should be used in CIP reinforced concrete box culverts.

The instrumentation plan developed to gather data from a CIP reinforced concrete box culvert in the field, and the test procedures for testing the concrete from the culvert are presented in Chapter 5. The instrumentation for gathering stress, strain, and temperature data as well as the instrumentation for detecting cracks and monitoring crack widths is described. The test procedures used in obtaining creep, drying shrinkage, compressive strength, tensile strength, modulus of elasticity, maturity, setting times, quality control, and early-age restrained stress development data are also described.

Recommendations for transverse contraction joints and wingwall joints are proposed in Chapter 6.

A general summary of the research findings, conclusions, and recommendations given is included in Chapter 7.

Chapter 2

Literature Review

2.1 Introduction

Tensile stresses rise due to the restraint of concrete volume change effects and cause the concrete to crack when the tensile stress exceeds the tensile strength of the concrete (ACI 224 1995). Because of this, it is generally accepted that most reinforced concrete structures will crack even when they are well designed (Mehta and Monteiro 2006). Humidity and thermal cycles cause concrete volume changes that make cracking inevitable (Mehta and Monteiro 2006; ACI 224 1995). These cracks can be an indication of the total magnitude of the distress, or they can be indications of more significant problems. The implications are dependent on the type of cracking and the function of the affected structure. For example, cracks that are not a problem in buildings could be detrimental to a structure that must retain water. (ACI 224 2007)

Concrete culverts are no exception to this. They typically experience longitudinal and transverse cracks between joints. Wide cracks can have the same effect as an open joint in a culvert. They can result in damage to the culvert by allowing the backfill material around the culvert to erode away. This can lead to alignment problems between the connecting barrels and to problems due to differential settlement. (AASHTO 2010a) Because of the problems that cracks can cause culverts, it is important to understand the causes of cracking, what can be done to prevent it, and how they can be repaired.

2.2 Early-Age Cracking Mechanisms

2.2.1 Thermal Stresses

Objects in the solid phase tend to contract when they are cooled, and expand when they are heated. The amount of expansion or contraction (strain) is a function of the coefficient of thermal expansion of a material and the temperature change. (Mehta and Monteiro 2006) Cracking can occur when restrained concrete goes through these temperature-related expansion and contraction phases (Bernander 1998).

The expansion comes from the heat produced by cement hydration (Mehta and Monteiro 2006). These cracks generally occur within one to a few days after the concrete has been poured and are usually only surface cracks. They also tend to close after the concrete has cooled and contracted. (Bernander 1998)

The shrinkage strain that occurs due to the concrete cooling (and contracting) tends to be significant (Mehta and Monteiro 2006). Shrinkage strains can lead to tensile stresses that usually cause through cracks if the contraction is at least partially restrained (Mehta and Monteiro 2006; Bernander 1998). They can form as soon as a few weeks after the concrete has been poured, or they can occur as late as years later. The through cracks usually remain permanently open, and can affect durability. (Bernander 1998) Figure 2-1 illustrates a through crack in a culvert.

Figure 2-1: Through Crack in a Culvert in the Anniston East Bypass (AEB) Project (149+60)

The tensile stresses that can result from thermal shrinkage strains are a function of the degree of restraint, modulus of elasticity, the stress relaxation due to creep, coefficient of thermal expansion, and the temperature change. If the thermal shrinkage strain is restrained, tensile stresses develop. However, if the shrinkage is unrestrained, stresses do not develop. Also, if the modulus of elasticity of the concrete is low, the tensile stress in the concrete will be lower. Stress relaxation due to creep also works to decrease the amount of stress experienced. Concrete that creeps more will have more stress relaxation. (Mehta and Monteiro 2006)

The coefficient of thermal expansion defines how one degree of temperature change affects the change in a unit length of unrestrained concrete (Mehta and Monteiro 2006). Research from Suh et al. (1992) showed that using limestone aggregate (low coefficient of thermal expansion) in concrete reduced the widths of the cracks in reinforced concrete pavements when compared to pavements that used siliceous river gravel (high coefficient of thermal expansion). Therefore, using aggregate with a low coefficient of thermal expansion can help to reduce the tensile stresses experienced (Mehta and Monteiro 2006). The temperature change for thermal deformations is defined as follows by Schindler (2002):

$$\Delta T = T_{zero-stress} - T_{\min}$$ **Equation 2-1**

where,

ΔT = concrete temperature change (°F),

$T_{\min}$ = minimum concrete temperature (°F), and

$T_{zero-stress}$ = concrete zero-stress temperature (°F).

Schindler (2002) also found that $T_{zero-stress}$ was between 92% and 94% of $T_{\max}$, and concluded that it should be approximated as follows:

$$T_{zero-stress} = 0.93 T_{\max}$$ **Equation 2-2**

where,

$T_{\max}$ = maximum concrete temperature (°F).

$T_{zero-stress}$ is used instead of $T_{\max}$ because, as Figure 2-2 illustrates, the concrete is in compression after final setting has occurred until the zero-stress state is reached. After the zero-stress state is reached, the concrete then first develops tensile stresses. Therefore, the maximum temperature that corresponds to the rise in tensile stresses that induce cracking is $T_{zero-stress}$. The lowest concrete temperature that will be experienced by the member could occur years after the concrete is cast, as presented in Figure 2-3. (Schindler 2002)

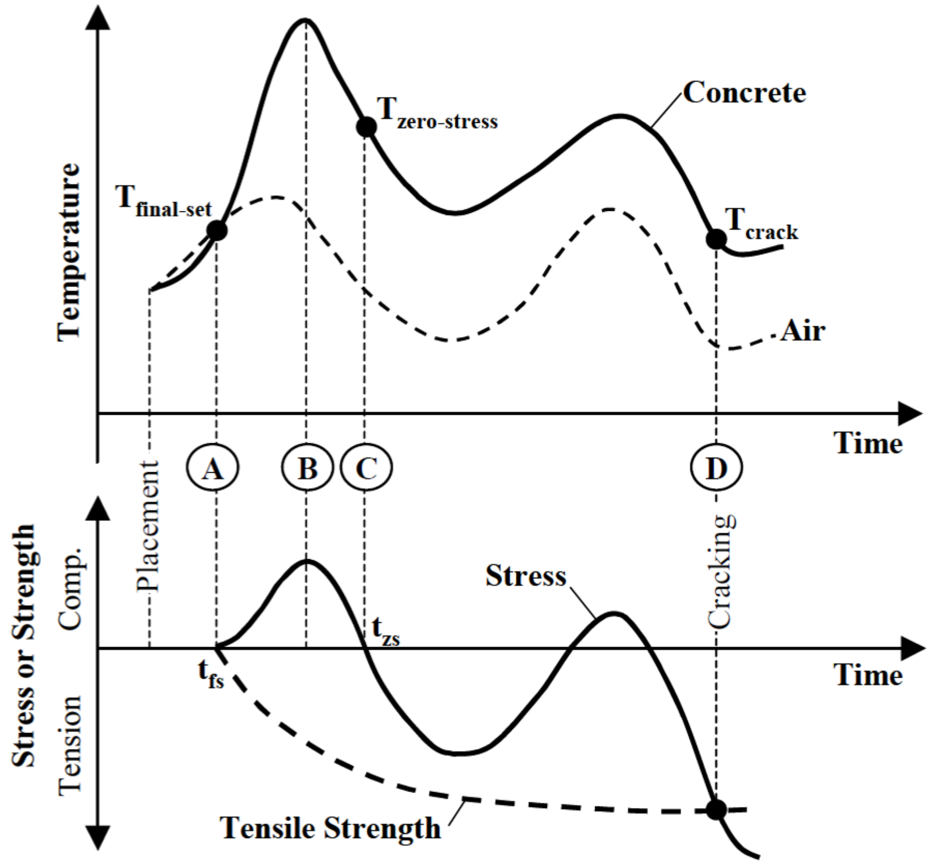


Figure 2-2: Development of Early-Age Thermal Stresses (Schindler 2002)

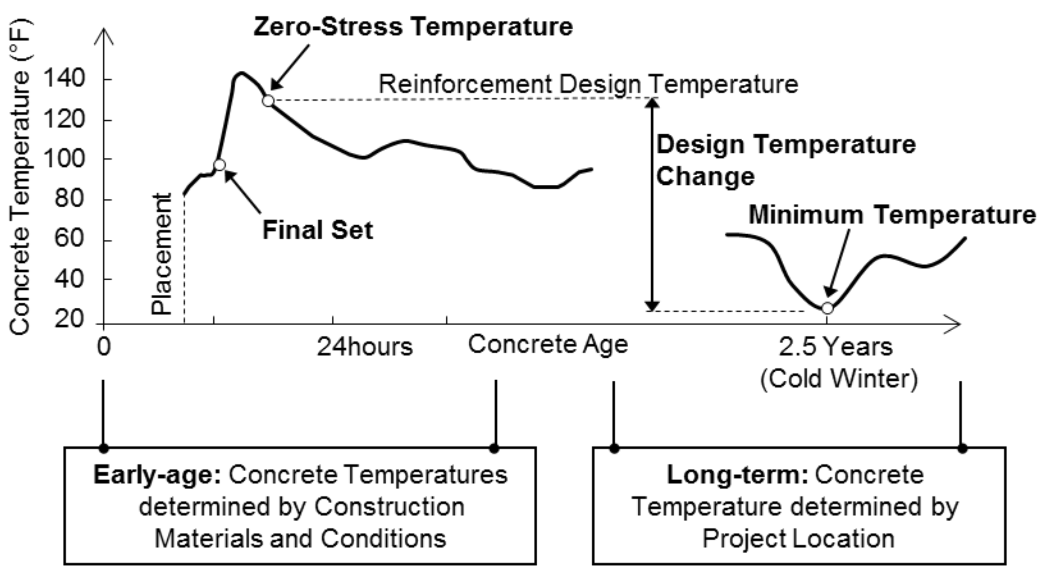


Figure 2-3: Concrete Thermal Deformation Temperature Change (Schindler 2002)

Research from Suh et al. (1992) also showed that the ambient temperature at the time of concrete placement had an effect on early-age cracking. They found that continuously reinforced concrete (CRC) pavement sections placed in the summer had wider crack widths than did pavement sections placed in the winter. This finding is illustrated in Figure 2-4. This was due to the high ambient temperatures in the summer increasing the rate of concrete hydration. When concrete hydrates at a higher rate, the temperature rise accelerates, which increases the zero-stress temperature. A large temperature differential then results when the ambient and concrete temperatures drop during the night after placement. In contrast, the temperature differential is smaller during winter construction due to retarded concrete hydration. They also found similar temperature differential results when comparing the time of day that CRC pavement sections were placed during the summer. The maximum concrete temperature from cement hydration, which typically occurs hours after placement, was greatly affected by the ambient temperature that coincided with it. The time of peak concrete temperature for concrete placed on a summer morning usually coincides with the peak ambient temperatures of the day. If placement occurs during a summer afternoon or night, the peak concrete temperature coincides with the lower ambient temperatures of the evening or night. As presented in Figure 2-5, it was found that placing the concrete in the morning increased the temperature rise (and therefore also the temperature differential) in the concrete, and it was found that it was preferable to place concrete in the afternoon or night during the summer. The time of concrete placement was found to have little effect during winter concrete placement. (Suh et al. 1992)

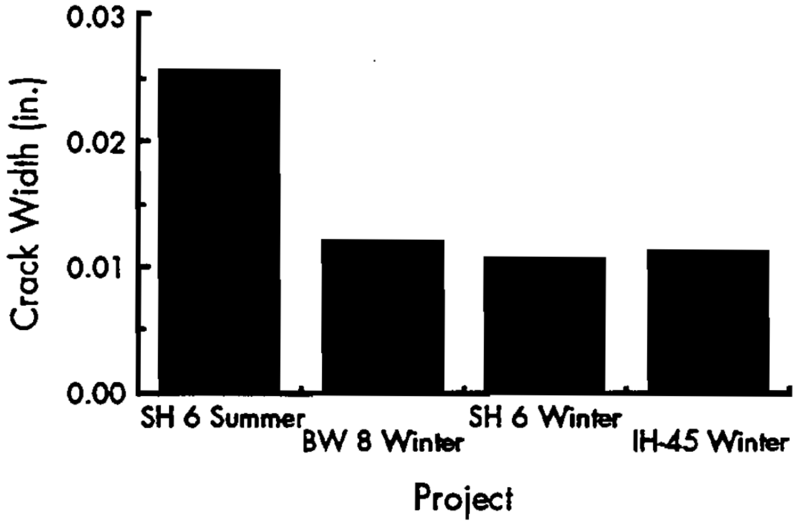


Figure 2-4: Effect of Placement Season on Crack Width (Suh et al. 1992)

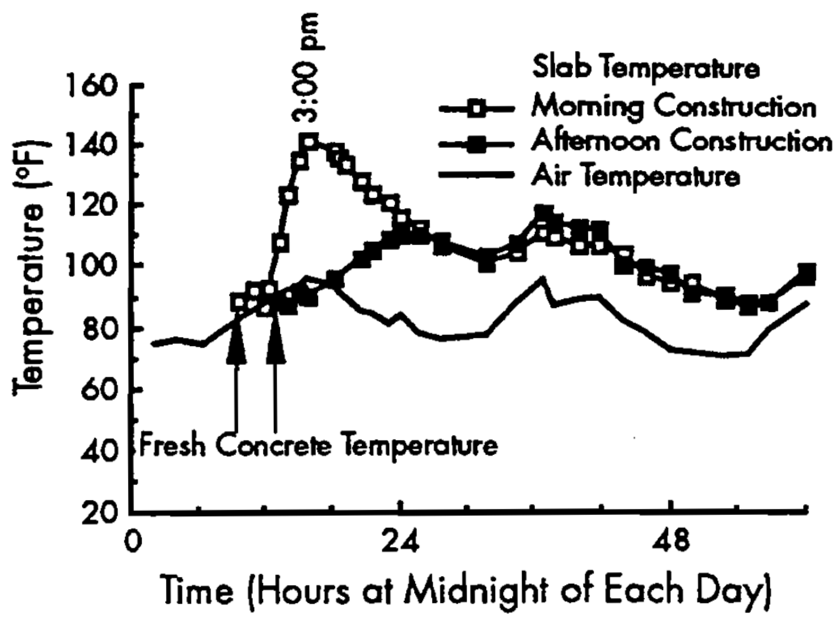


Figure 2-5: Temperature Variations for Different Placement Times (SH 6 Summer) (Suh et al. 1992)

2.2.2 Plastic Shrinkage

Plastic shrinkage is a problem in concrete before setting occurs. It is a concern in hot, windy areas and usually associated with concrete slabs (Folliard et al. 2009; Mehta and Monteiro 2006). Plastic shrinkage happens as a result of water evaporating from the surface of fresh concrete quicker than bleedwater can replenish it. The underlying concrete acts as a restraint for the surface layer and causes tensile stresses to develop in the contracting surface. (ACI 224 2007) The tensile strength of the concrete in its plastic state is negligible. Therefore the tensile stresses experienced exceed the tensile strength and plastic shrinkage cracking occurs. (Folliard et al. 2009) The cracks formed are generally shallow but can become much deeper with time (ACI 224 2007). An example of a plastic shrinkage crack is shown in Figure 2-6. They typically are only 1 to 2 in. (25 to 50 mm) deep and 1 to 6.5 ft (0.3 to 2.0 m) apart (Mehta and Monteiro 2006). Also, plastic shrinkage cracks can run parallel to each other or follow a polygonal pattern. They can be relatively wide (up to 1/8 inch [3.2 mm]), and vary in length. (ACI 224 2007)

Figure 2-6: Plastic Shrinkage Crack in the Anniston East Bypass Project (175+70)

Concrete that has a low tendency to bleed (such as concrete with silica fume) has an increased susceptibility to plastic shrinkage cracking (ACI 224 2007). This is because the required evaporation rate for plastic shrinkage to occur is much lower than it is for concrete with a greater tendency to bleed (Folliard et al. 2009; ACI 224 2007).

The rate of evaporation of moisture in fresh concrete is key to the occurrence of plastic shrinkage. It is a function of concrete temperature, air temperature, wind velocity, and relative humidity. High concrete temperatures, high air temperatures, high wind velocities, and low relative humidity all increase the evaporation rate and plastic shrinkage. (ACI 224 2007) When the evaporation rate exceeds 0.2 lb/ft^2 per hour (1 kg/m^2 per hour), plastic shrinkage can become a problem and precautionary actions should be taken (Mehta and Monteiro 2006).

2.2.3 Autogenous and Chemical Shrinkage

Chemical shrinkage is due to the products of cement hydration taking up less volume than the initial water and cement did (Jensen and Hansen 2001). After the concrete has initially set, the concrete paste cannot change shape as much as it could before. As a result, hydration, and therefore chemical shrinkage, continues by forming voids in the microstructure of the paste. Most chemical shrinkage happens internally and is not visible in the external geometry of the concrete. (Kosmatka et al. 2002)

The American Concrete Institute (2010) defines autogenous shrinkage as “a change in volume produced by continued hydration of cement, exclusive of effects of applied load and change in either thermal condition or moisture content.” It is often too small to measure in normal strength concrete, because the concrete generally has enough water for the concrete to fully hydrate (Holt 2001). However in high-performance concrete (HPC), the autogenous shrinkage can be significant due to its low water-cement ratio and high cement content (Holt

2001; Mehta and Monteiro 2006). These conditions can cause autogenous shrinkage in the form of self-dessication to occur when there is not enough water available to hydrate all of the cement (Holt 2001; Folliard et al. 2009). Self-desiccation is when the concrete takes water from its own pore cavities for hydration purposes. It is a form of internal drying, and it lowers the internal relative humidity of the concrete. (Holt 2001) Typical magnitudes of autogenous and chemical shrinkage are illustrated in Figure 2-7.

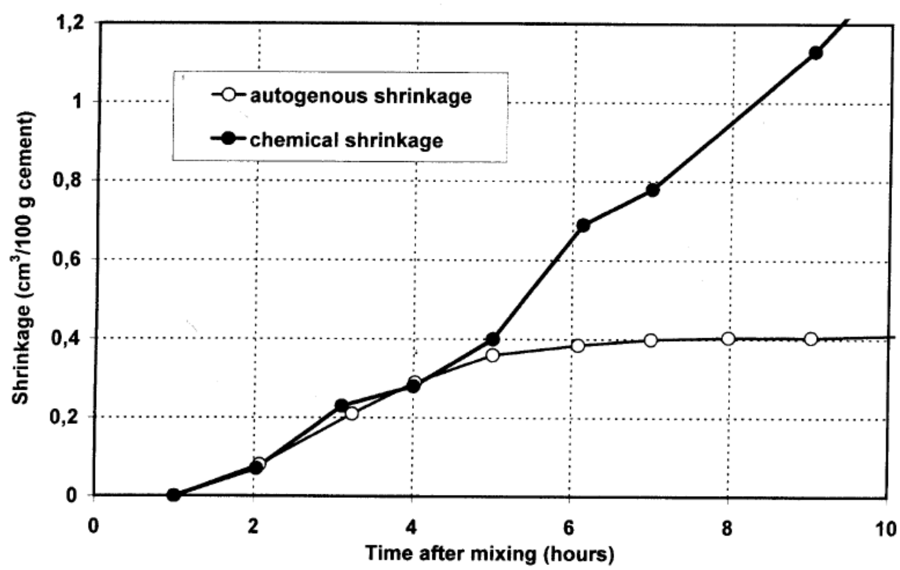


Figure 2-7: Relationship between Autogenous Shrinkage and Chemical Shrinkage of Cement Paste at Early Ages (Hammer 1999)

Cracking due to autogenous shrinkage is also due to the concrete being restrained when shrinkage occurs. Microcracks can form and connect to form a continuous crack network. This can greatly diminish concrete's strength and durability resistance, as well as causing aesthetic problems. (Jensen and Hansen 2001)

2.2.4 Drying Shrinkage

The catalyst for drying shrinkage is the same as for plastic shrinkage: loss of internal moisture from the cement paste (ACI 224 2007). However, in the case of drying shrinkage, the water lost was absorbed in the hydrated cement paste (Mehta and Monteiro 2006). The major difference is that drying shrinkage occurs in hardened concrete and plastic shrinkage occurs in fresh concrete. Moisture is lost first from the largest pores where it is most loosely held, and it is then lost from smaller and smaller pores. Water lost from pores that are smaller than 50 nanometers is responsible for drying shrinkage. (Folliard et al. 2009) Water in pores larger than 50 nanometers is considered free water and is not considered responsible for any drying shrinkage (Mehta and Monteiro 2006).

The magnitude of drying shrinkage is affected by the aggregate used, the cement and water contents of the cement paste, the geometry of the concrete member, humidity, and time (ACI 224 2007; Mehta and Monteiro 2006). Research performed by Pickett (1956) showed that increasing the aggregate content will reduce the amount of drying shrinkage experienced. Increasing the water-cement ratio (when the concrete has a fixed cement content) increases the amount of drying shrinkage, as does increasing the cement content at a fixed water-cement ratio. Concrete members that are smaller, (or have a shorter path for water to leave the concrete) tend to experience drying shrinkage quicker. (Mehta and Monteiro 2006) Research by Hansen and Almudaiheem (1987), see Figure 2-8, showed that no drying shrinkage occurs when the relative humidity is at 100%, and that the amount of drying shrinkage experienced increased as the relative humidity decreased. This is because more moisture is lost from the concrete as the relative humidity lowers (Mehta and Monteiro 2006).

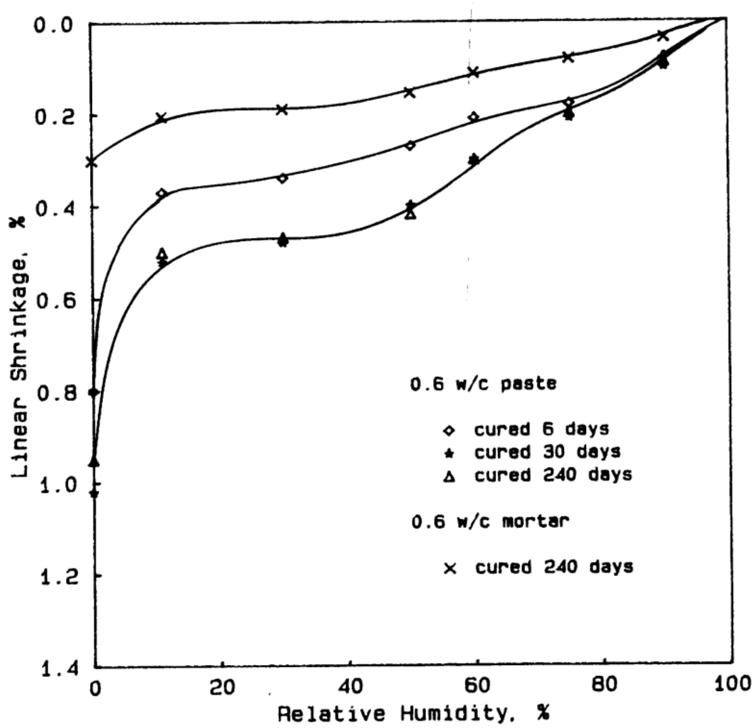


Figure 2-8: Influence of Relative Humidity on Ultimate Drying Shrinkage (Hansen and Almudaiheem 1987)

Drying shrinkage by itself does not cause cracking. The restraint of the concrete during its shrinkage-induced volume change causes the tensile stresses that cause cracking. Figure 2-9 illustrates this concept. The size of the tensile stresses experienced are a function of the degree of restraint, amount of shrinkage, rate of shrinkage, the modulus of elasticity, and the magnitude of creep. (ACI 224 2007) The modulus of elasticity and creep (through stress relaxation) affect drying shrinkage tensile stresses in the same way that they affect thermal tensile stresses (Mehta and Monteiro 2006). In mass concrete pours, tensile stresses arise when the amount of shrinkage on the exterior is greater than in the interior. This causes cracks on the surface that can propagate deeper into the concrete as time passes. (ACI 224 2007)

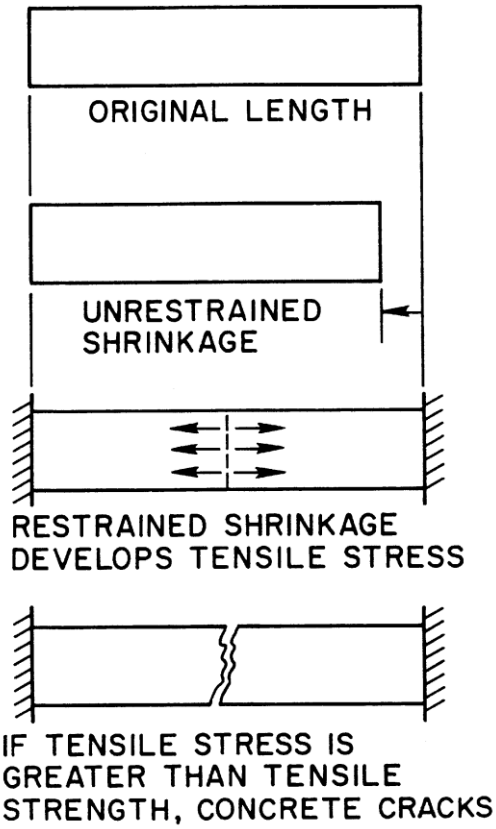


Figure 2-9: Drying Shrinkage Cracking Illustration (ACI 224 2001)

Alligator (or craze) cracking located on slabs or walls is an example of drying shrinkage cracking (ACI 224 2007). Alligator cracking is defined as irregular fine cracks on the exterior of concrete, and it is a result of the surface portion of concrete having a higher water-cement ratio than its inner portion (ACI 201 1992; ACI 224 2007). An example of alligator cracking is shown in Figure 2-10.

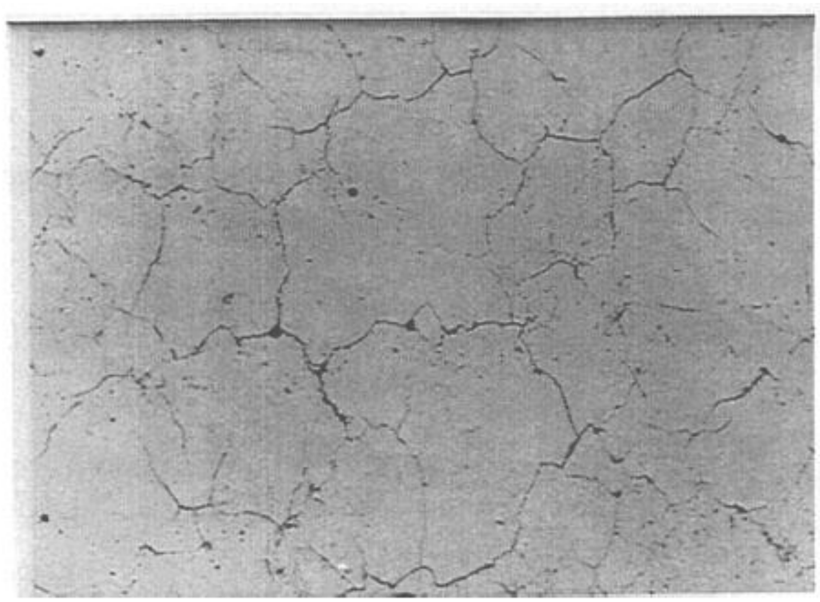


Figure 2-10: Alligator or Craze Cracking (ACI 201 1992)

2.3 Long-Term Durability

2.3.1 Corrosion

The causes of reinforcement corrosion can be attributed to different things. An electrical current is necessary for it to occur. The electrical current can happen because of potential differences between an anode and a cathode in reinforcing steel. The positively charged iron (Fe^{2+}) ions from the steel move from the anode to the cathode where they react with negatively charged hydroxide ions (OH^-) (from water and oxygen) to form iron oxide or rust. (Pincheira et al. 2008) This process is illustrated in Figure 2-11.

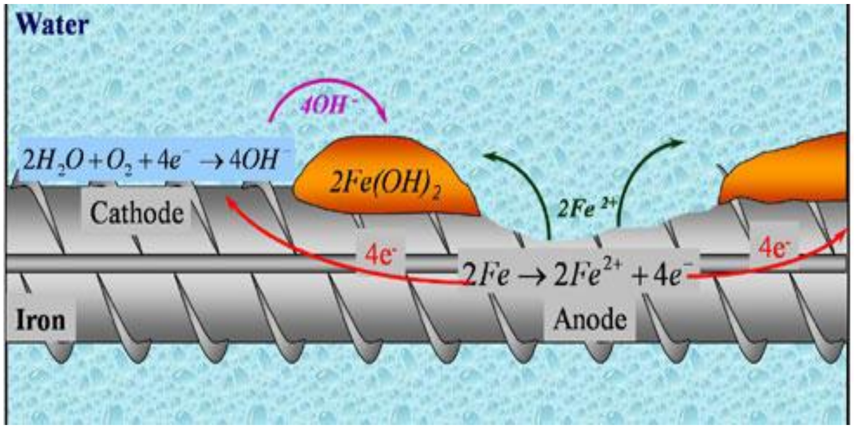


Figure 2-11: Reinforcement Corrosion Process (PCA n.d.)

The presence of chlorides, which are present in deicing salts and seawater, is the major contributor to corrosion problems. Bridge decks provide a good example of this. Negatively charged chlorides penetrate concrete and concentrate around the reinforcement to form negatively charged (anodic) sections. The other sections are thus more positive (cathodic) by comparison. This produces the electrical current necessary for corrosion to occur. (Pincheira et al. 2008) Another way to explain how chlorides aid in corrosion is the fact that steel reinforcement has an initial oxide film over it from when it was produced. This oxide film is stabilized in the alkaline environment of concrete. This film protects the rebar from corrosion. Liquids containing chlorides permeate the concrete and they help to wear away this protective film. (Kuennen 2010)

Carbonation is yet another way that reinforcement can be attacked. This occurs when carbon dioxide in the air reacts with calcium hydroxide in concrete to form carbonic acid. The acid reduces the pH of concrete and produces a less alkaline environment. Because of the reduction in alkalinity, the protective film around the reinforcement becomes unstable and breaks away. (Mathew 2006) This increases the susceptibility of the reinforcement to corrosion (Kuennen 2010).

Concrete cracking is a dangerous side effect of reinforcement corrosion. The hydroxides and iron oxides produced from corrosion reactions take up more space than the uncorroded steel did (Pincheira et al. 2008). This increases the stresses in concrete around the reinforcement bars and can lead to localized cracking around the corroded section (ACI 224 2007). The localized cracks can propagate into long longitudinal cracks (cracks running parallel to the reinforcing bars) and can result in delamination (when a surface layer of concrete separates from the reinforcement) (ACI 224 2007; Mathew 2006). The longitudinal cracks also make it easier for corrosive agents (moisture, oxygen, and chlorides) to reach the reinforcement, thus allowing the corrosion effects to get worse. Therefore, wider cracks lead to greater corrosion because more of the bar is exposed to the corrosive agents. (ACI 224 2007) Cracks with a width greater than 0.006 in. (0.2 mm) are considered to be a problem in when concrete under service loads is exposed to seawater, and cracks greater than 0.007 in. (0.2 mm) are a problem when exposed to deicing chemicals (ACI 224 2001). Cracks that are transverse to the reinforcing steel generally do not enhance the effects of corrosion if the concrete has a low permeability (ACI 224 2007). An example of corrosion damage is shown in Figure 2-12.

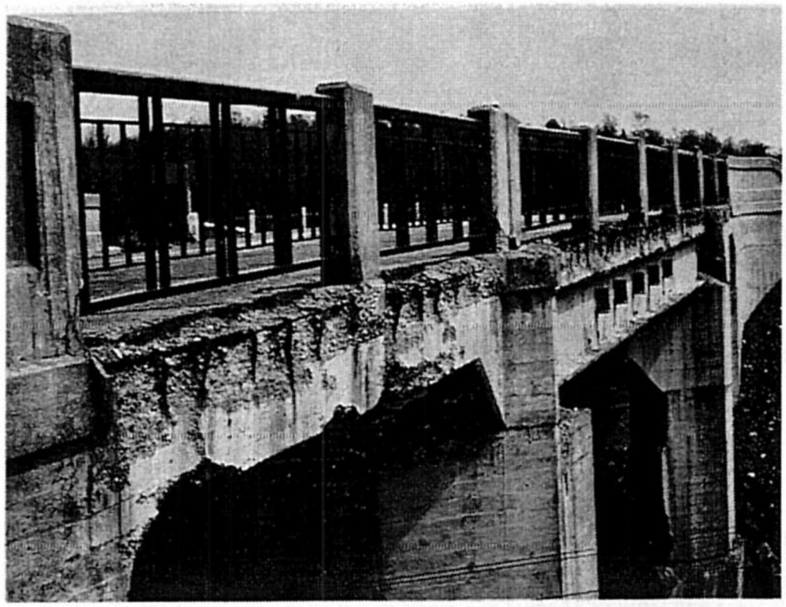


Figure 2-12: Corrosion Damage (ACI 201 1992)

2.4 Other Causes of Concrete Distress

2.4.1 Inadequate Curing

Inadequate curing can greatly increase the amount of cracking in a structure. If concrete is not allowed to cure long enough, the amount of shrinkage experienced will increase, and it will happen at a point when the concrete is low in strength. Inadequate curing will also lead to decreased durability and long-term strength. (ACI 224 2007) Because of this, it is important that concrete be cured in a way so that it is protected from low temperatures and conditions that could cause early drying. These conditions could cause cracking if they are not taken into account. It is also necessary to make sure the concrete is cured in a way so that it has adequate strength. (ACI 224 2001)

Allowing concrete to cure longer can mitigate cracking. Stress relaxation (through creep) and the concrete being allowed to adjust to the restrained stresses over the curing period make this possible. A wet curing period of at least 7 days is recommended. (ACI 224 2001)

2.5 Crack Control

2.5.1 Reinforcement

The percentage of steel in reinforced concrete can also have an effect on the crack width and spacing. Research by McCullough and Dossey (1999) showed that increasing the reinforcement percentage in CRC pavements decreased the crack spacing. See Figure 2-13 for their findings. Research from Suh et al. (1992) showed that increasing the reinforcement percentage in CRC pavements decreased the crack widths experienced. Their findings are presented in Figure 2-14.

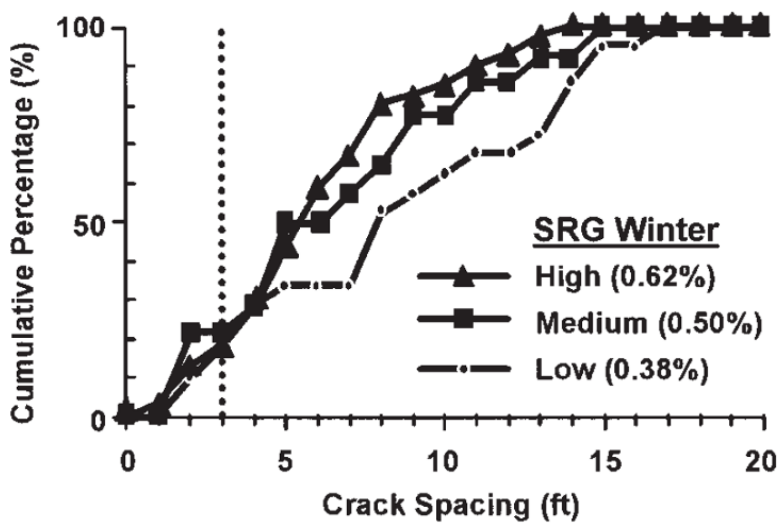


Figure 2-13: Effect of Steel Percentage on Crack Spacing (McCullough and Dossey 1999)

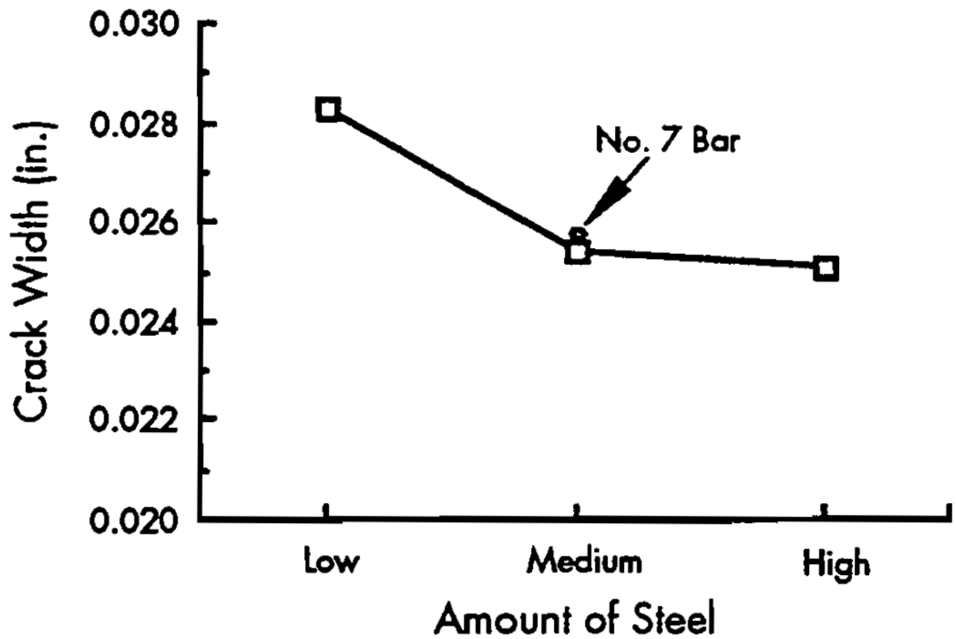


Figure 2-14: Effect of Longitudinal Steel Design on Crack Width (Suh et al. 1992)

Examining the extreme case of unreinforced and reinforced concrete further confirms the effect of reinforcement percentage on cracking spacing. Unreinforced concrete tends to have cracks spaced at larger intervals than reinforced concrete. This is because the steel provides restraint when the concrete expands or contracts. The steel percentage can be altered to achieve the desired crack spacing. (McCullough and Dossey 1999)

ACI 350 (2001) recommends that a minimum temperature and shrinkage reinforcement percentage of between 0.0030 and 0.0050 be used for environmental structures depending on the length between joints in a structure. Table 2-1 summarizes their recommendations. ACI 224 (2001) recommends that a reinforcement ratio of at least 0.0060 be used to control shrinkage cracking, and that the minimum reinforcement ratio of 0.0018 for structural slabs and 0.0020 for walls specified by ACI 318 (2011) is insufficient in members with a high degree of restraint. Increasing the reinforcement percentage in slabs that are significantly restrained is supported in

Section R7.12.1.2 of the commentary of ACI 318 (2011), which states, "Where structural walls or columns provide significant restraint to shrinkage and temperature movements…, it may be necessary to increase the amount of slab reinforcement required."

Table 2-1: Minimum Temperature and Shrinkage Reinforcement Ratios for Environmental Structures (ACI 350 2001)

Length between movement joints, ft	Minimum shrinkage and temperature reinforcement ratio	
	Grade 40	Grade 60
Less than 20	0.0030	0.0030
20 to less than 30	0.0040	0.0030
30 to less than 40	0.0050	0.0040
40 and greater	0.0060*	0.0050*

*Maximum shrinkage and temperature reinforcement where movement joints are not provided.
Note: When using this table, the actual joint spacing shall be multiplied by 1.5 if no more than 50% of the reinforcement passes through the joint.

The size of the reinforcement bars also has an effect on crack spacing. When the steel percentage was kept constant in CRC pavements, McCullough and Dossey (1999) found that larger bar sizes led to increased crack spacing. Figure 2-15 illustrates their research. This occurred because larger reinforcement bars have a larger bond area between the concrete and reinforcement (McCullough and Dossey 1999).

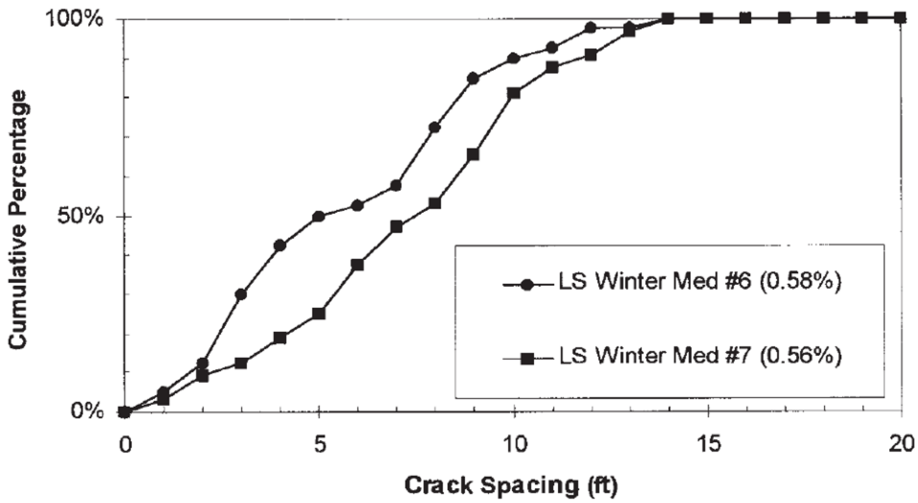


Figure 2-15: Effect of Reinforcement Bar Size on Crack Spacing (McCullough and Dossey 1999)

2.5.2 Joints

Jointing is an important aspect of crack control and construction (Kosmatka et al. 2002). Joints allow for concrete to be placed in sections instead of continuously and can be used to relieve stresses due to concrete volume changes. Also, joints are viewed as intentional cracks by some engineers and can be used as weakened planes to ensure that cracks occur in places that are of minor importance. (ACI 224 1995)

Joints serve a specific purpose when used in culverts (or tunnels), slabs-on-grade, or walls. Transverse joints in cast-in-place (CIP) tunnels, and therefore in culverts, serve the purpose of reducing shrinkage cracking and simplifying construction. Longitudinal tunnel joints segment the cross section of a CIP tunnel into sections. Their location depends on the cross section and the order of concrete placement. Joints in slabs-on-grade serve the purpose of allowing the slab to move. They also make sure that the slab is aesthetically pleasing by providing a relatively crack-free slab. The joints cause the cracks to form along designated planes. In walls, the base slab acts as restraint for walls that crack when the wall deforms and exceeds its tensile strength. Therefore, joints in walls serve the same purpose as they do in slabs-on-grade by allowing movement and by controlling cracking. Construction joints can also allow for the wall to be placed in sections. (ACI 224 1995)

The three types of joints that will be reviewed are construction joints, contraction joints, and expansion joints. Joint sealants, joint filler, and waterstops used in these joints will also be evaluated.

2.5.2.1 Construction Joints

Construction joints separate areas of concrete that have been placed at different times (Kosmatka et al. 2002). These joints may also serve the purpose of a construction or expansion joint, where it allows the concrete to expand or contract. Construction joints can also allow no movement at all by being bonded to a previously placed section. (Kosmatka et al. 2002) If construction joints are placed at points of high stress, the joints can open (ACI 224 2007). Construction joints should be put in places that affect structural integrity the least and that are consistent with the structure's appearance (ACI 224 1995).

Bonded and butt joints are two typical transverse construction joints. Bonded joints should be used if the concrete will have time to harden before the next placement. Tie bars may be used in bonded construction joints to restrain movement. Continuous reinforcement, which is typical in walls that must maintain flexural and shear continuity, may also be used in bonded joints. Figure 2-16 illustrates a bonded construction joint with continuous reinforcement.

Butt joints are typically used in thin slabs-on-grade that will not carry heavy loads. However, they can be used in thick slabs-on-grade that will carry heavy loads, but keys or dowel bars will be needed to transfer load from one placement to the next. (ACI 224 1995) These plain, keyed, or dowelled butt joints may also be used as contraction joints (Kosmatka et al. 2002). Figure 2-17A illustrates a butt joint and Figure 2-17B illustrates a butt joint with a dowel bar.

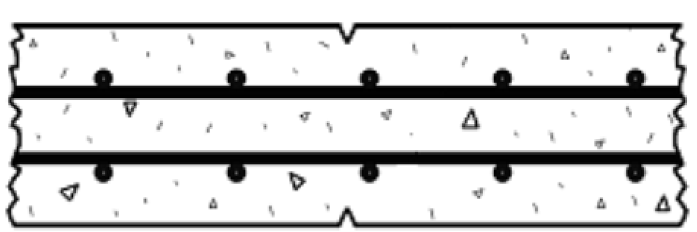


Figure 2-16: Construction Joint with Continuous Reinforcement (ALDOT 2010)

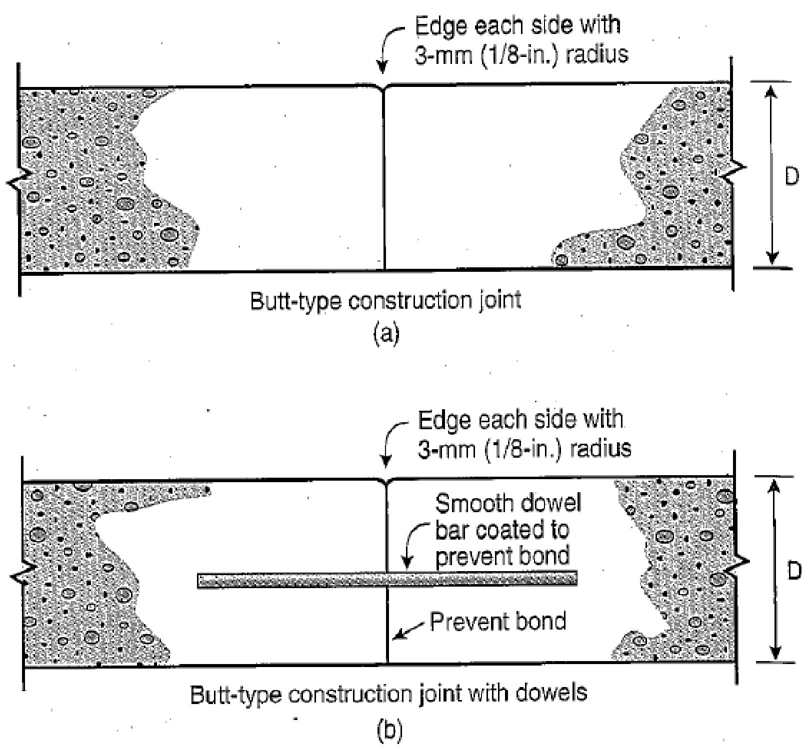


Figure 2-17: Illustrations of a: (A) Butt Construction Joint and (B) Butt Construction Joint with Dowel Bars (Kosmatka et al. 2002)

Longitudinal construction joints may also be used in box culverts and similar structures (such as tunnels). These joints segment the cross section of the culvert or tunnel into different parts. These construction joints may be sealed by the bond between the adjoining concrete members (e.g. top slab and wall or wall and base slab). Also, a waterstop may be used in the joint if a watertight seal is necessary. (ACI 224 1995) The possible locations of longitudinal construction joints in an ALDOT culvert are shown in Figure 2-18.

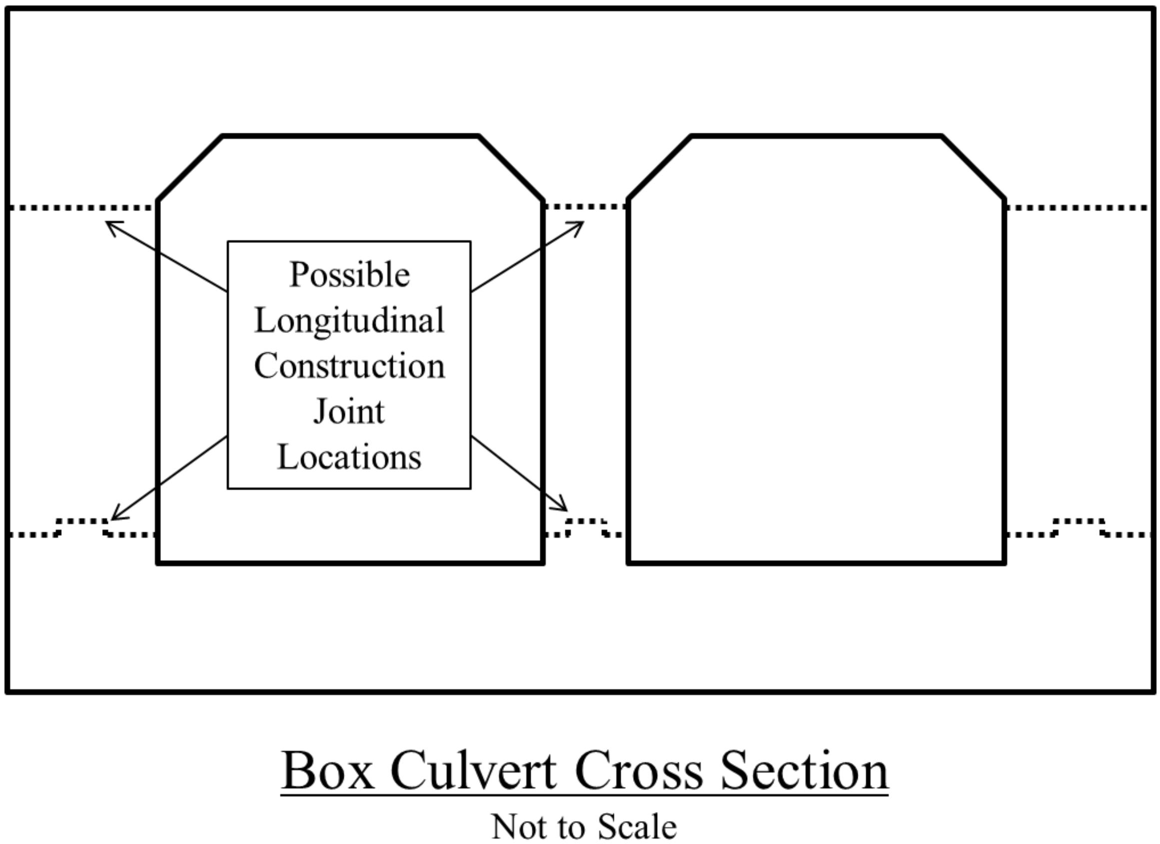


Figure 2-18: Possible Longitudinal Construction Joint Locations in an ALDOT Box Culvert

2.5.2.2 Contraction Joints

Contraction joints serve the purpose of allowing the concrete to move and allow for controlled cracking due to shrinkage and thermal stresses to occur (Kosmatka et al. 2002). They also allow for these stresses to be relieved (ACI 224 1995). Figure 2-19 illustrates how tensile stresses in a wall from restrained shrinkage can cause cracking and how a contraction joint can alleviate those stresses.

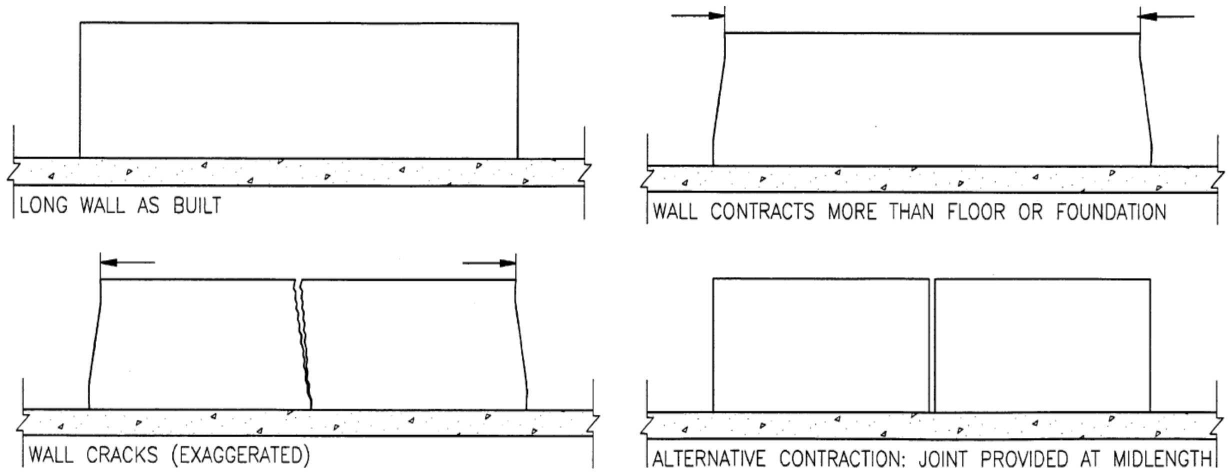
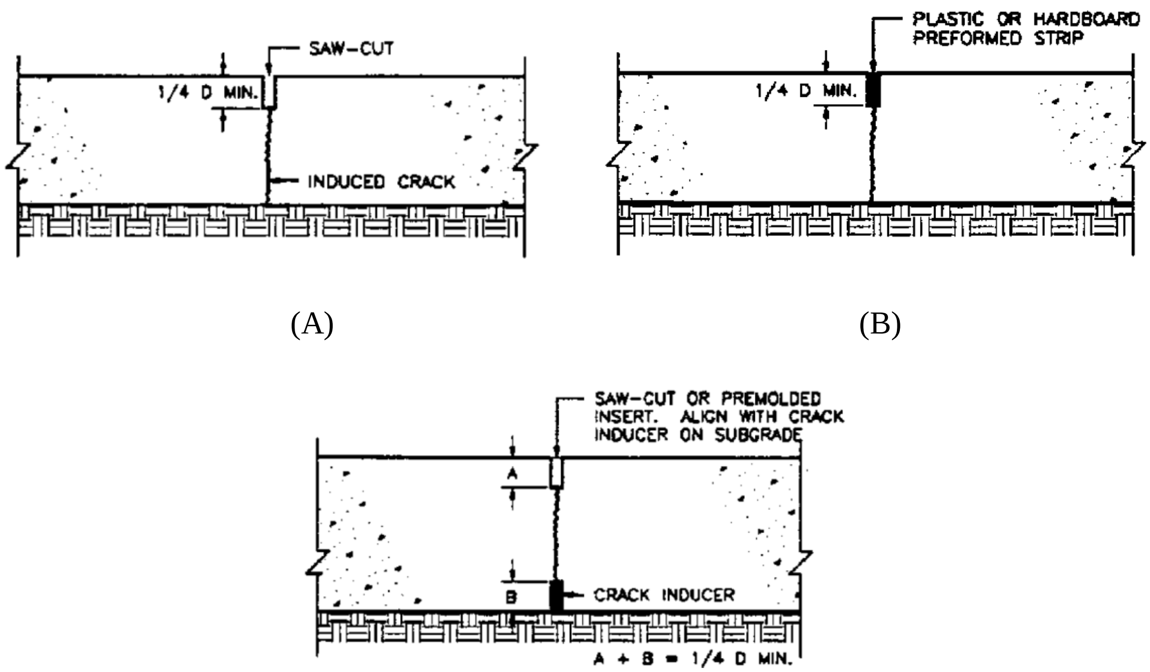


Figure 2-19: Contraction Joint Concept (ACI 224 1995)

Contraction joints are formed by saw cutting, hand tooling, by preformed inserts, or between adjacent concrete placements. Joints in slabs-on-grade are formed using all of these methods while joints in walls are typically only formed by using premolded inserts. Saw cutting involves cutting a groove into hardened concrete with a saw. The groove should be cut soon after the concrete has hardened. (ACI 224 1995) Figure 2-20a illustrates a saw-cut contraction joint. Creating hand-tooled contraction joints involve creating a groove using a hand tool (Kosmatka et al. 2002). Preformed joints are formed by putting wood, rubber, metal, or plastic strips into concrete before it is finished to create grooves. The grooves for all methods of forming contraction joints should be at least ¼ of the concrete thickness in order to create a sufficient plane of weakness. Figure 2-20b illustrates a contraction joint formed with a preformed strip. If slabs-on-grade are so thick that hand-tooling or inserting a preformed strip is troublesome, a premolded strip can also be put in the bottom part of the sla. The combined depth of the inserts should be greater than ¼ of the slab thickness. This type of joint is illustrated in Figure 2-20c. A similar method is used in walls for providing grooves on the interior and exterior faces. (ACI 224 1995)



(C)

Figure 2-20: Illustrations of a: (A) Saw Cut Contraction Joint, (B) Contraction Joint with a Premolded Insert, and (C) Contraction Joint in a Thick Slab (ACI 224 1995)

When load transfer is required across a joint, keys may need to be formed for contraction joints. This can be done by placing a full-depth preformed key in the slab when it is placed. Beveled wood strips or metal forms can also be used to form the key. (ACI 224 1995)

Contraction joints in slabs-on-grade subdivide the full slab into smaller segments. Because of this, joints must be able to transfer vertical loads to the adjacent slab segments. This can be achieved through aggregate interlock, keys, or by the use of dowel bars. (ACI 224 1995)

Aggregate interlock can be effective if the loads experienced are light (joints in slabs-on-grade have performed well with loads of up to 5,000 lb [22 kN]). However, if the loads are repetitive the joint can deteriorate. The type of subgrade the slab is built on can also determine the effectiveness of aggregate interlock. Sandy soils tend to provide more support than some silt and clay soils. The type of aggregate used in the concrete affects aggregate interlock too. Crushed aggregate transfers loads better than natural gravel does, and coarse aggregate works better than fine aggregate does. Crack widths should be smaller than 0.035 in. (0.89 mm) for aggregate interlock to work effectively. (ACI 224 1995)

Keyed joints are also effectively transfer loads. The keys are formed so that there will be a tongue-and-groove at the joint. Figure 2-21 shows an illustration of a keyed tongue-and-groove contraction joint. Due to the joint being beveled, load transfer depends on there being little movement at the joint. (ACI 224 1995) ACI 302 (2004) does not recommend that keyed joints be used in slabs-on-grade when heavy loads are transferred. Their reasoning is that the key and keyway do not remain in contact when the slab shrinks, which leads to the deterioration of the concrete joint edges (ACI 302 2004).

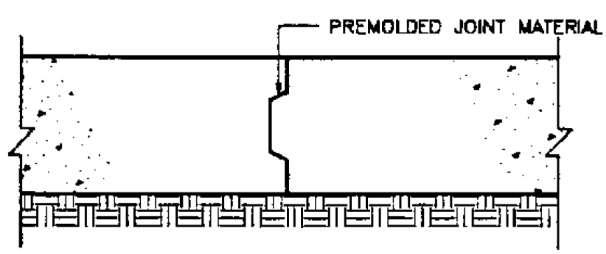


Figure 2-21: Keyed Tongue-and-Groove Contraction Joint (ACI 224 1995)

Heavy loads may be too much for aggregate interlock and keys to be effective. Dowel bars can be used at joints in slabs that have heavy loads that need to be transmitted across the joint. Figure 2-22 shows an example of a contraction joint with a dowel bar. The dowel bars should be parallel to each other and level. Also, they should be located at mid-height of the joint in the slab. The dowel should not be bonded to the concrete on at least one side of the joint to allow for slab contraction/expansion. Bonding can be prevented using greased dowels or wrapping bond-breaking plastic around the dowels. The dowel bars should be smooth. Dowel bars are also used in wall contraction joints. (ACI 224 1995)

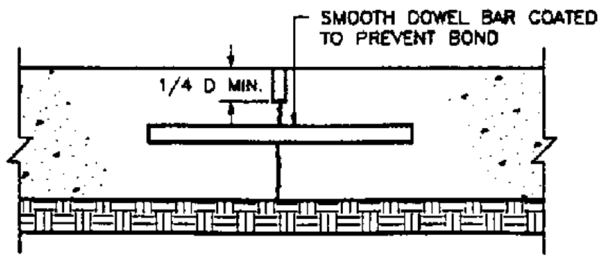


Figure 2-22: Doweled Contraction Joint (ACI 224 1995)

Sealants are sometimes necessary in contraction joints. Weather-resistant polyurethane or silicone can be used as sealants for contraction joints in walls. A waterstop may also be used to keep water from leaking through the joint. (ACI 224 1995)

There are different reinforcement options at contraction joints. Reinforcement can stop and not be continuous through the joint, some of the reinforcement can continue through the joint, or all reinforcement can continue through the joint. All of the reinforcement should continue through the joint only when it is necessary to maintain structural stability. (ACI 224 1995)

There are many different recommendations for contraction joint spacing. AASHTO (2010b) recommends that contraction joints be used throughout the length of a tunnel spaced approximately at 30 ft (9.1 m). Other recommendations compiled by ACI 224 (1995) for contraction joint spacing are shown in Table 2-2. The ACI 224-92 spacing recommendation is one times the wall height for high walls (typically higher than 12 ft [3.7 m]) and three times the wall height for short walls (typically smaller than 8 ft [2 m]) (ACI 224 1995). Recommendations for contraction joint spacing in slabs-on-grade compiled by Kosmatka et al. (2002) are shown in Table 2-3.

Table 2-2: Recommended Contraction Joint Spacing (ACI 224 1995)

Author	Spacing
Merrill (1943)	20 ft (6 m) for walls with frequent openings, 25 ft (7.5 m) in solid walls.
Fintel (1974)	15 to 20 ft (4.5 to 6 m) for walls and slabs on grade. Recommends joint placement at abrupt changes in plan and at changes in building height to account for potential stress concentrations.
Wood (1981)	20 to 30 ft (6 to 9 m) for walls.
PCA (1982)	20 to 25 ft (6 to 7.5 m) for walls depending on number of openings.
ACI 302.1R	15 to 20 ft (4.5 to 6 m) recommended until 302.1R-89, then changed to 34 to 36 times slab thickness
ACI 350R-83	30 ft (9m) in sanitary structures.
ACI 350R	Joint spacing varies with amount and grade of shrinkage and temperature reinforcement.
ACI 224R-92	One to three times the height of the wall in solid walls.

Table 2-3: Recommended Spacing (in Feet) for Contraction Joints in Slabs-on-grade (Kosmatka et al. 2002)

Slab thickness, in.	Maximum-size aggregate less than ¾ in.	Maximum-size aggregate ¾ in. and larger
4	8	10
5	10	13
6	12	15
7	14	18**
8	16**	20**
9	18**	23**
10	20**	25**

* Spacings are appropriate for slumps between 4 in. and 6 in. If concrete cools at an early age, shorter spacings may be needed to control random cracking. (A temperature difference of only 10°F may be critical.) For slumps less than 4 in., joint spacing can be increased by 20%.

** When spacings exceed 15 ft, load transfer by aggregate interlock decreases markedly.

2.5.2.3 Expansion Joints

Expansion joints are designed to allow concrete to expand without crushing and distorting the adjacent concrete (ACI 504 1990). They are also referred to as isolation joints. Expansion joints in walls are usually vertical joints that extend through the concrete between walls. They separate adjacent concrete and allow each to move freely. The movements at expansion joints could be due to the concrete expanding, applied loads, or differential movement. Temperature change is a major part of the expansion experienced in walls. Expansion joints can also be put in slabs-on-grade. However, expansion in slabs-on-grade is typically smaller than the initial shrinkage that the slab undergoes. Because of this, expansion joints are seldom needed. (ACI 224 1995)

The space or opening between concrete placements in expansion joints is created by joint filler. Compressible, elastic, and nonextruding joint filler, such as premolded mastic or cork, is typically used. The joint should extend the full height or width of the cross section. Figure 2-23 illustrates a typical expansion joint. (ACI 224 1995)

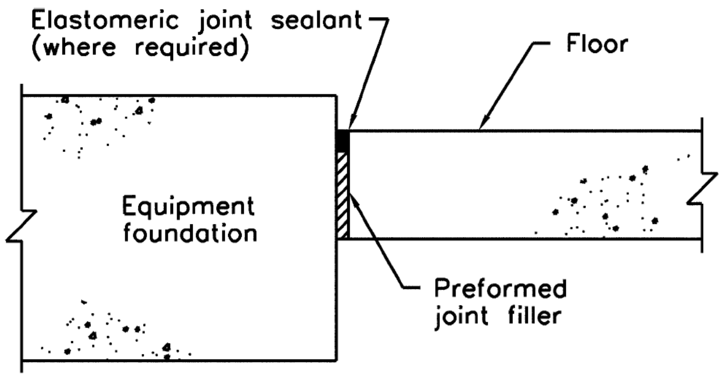


Figure 2-23: Typical Expansion/Isolation Joint (ACI 302 2004)

Lateral displacements at expansion joints sometimes need to be limited. This can be achieved by using dowel bars, steps, or keys. (ACI 504 1990)

There are many different recommendations for the appropriate expansion joint spacing. Typical expansion joint spacing in walls is from 200 to 300 ft (61.0 to 91.4 m). Also, expansion joints should be located at places where walls change direction, or where multiple walls come together from different directions. (ACI 224 1995) More contraction joint spacing recommendations compiled by ACI 224 (1995) are shown in Table 2-4.

Table 2-4: Recommended Expansion Joint Spacing (ACI 224 1995)

Author	Spacing
Lewerenz (1907)	75 ft (23 m) for walls.
Hunter (1953)	80 ft (25 m) for walls and insulated roofs, 30 to 40 ft (9 to 12 m) for uninsulated roofs.
Billig (1960)	100 ft (30 m) maximum building length without joints. Recommends joint placement at abrupt changes in plan and at changes in building height to account for potential stress concentrations.
Wood (1981)	100 to 120 ft (30 to 35 m) for walls.
Indian Standards Institution (1964)	45 m (≈ 148 ft) maximum building length between joints.
PCA (1982)	200 ft (60 m) maximum building length without joints.
ACI 350R-83	120 ft (36 m) in sanitary structures partially filled with liquid (closer spacings required when no liquid present.

2.5.2.4 Joint Sealants, Joint Fillers, and Waterstops

Joints can either be filled, sealed, or left open. Some contraction joints in floors used for industrial and commercial applications can be left open. This is because little movement will occur at these joints. However, a joint must be filled if it is exposed to moisture, must meet hygienic and dust-control specifications, or is subjected to small, hard-wheel vehicle traffic. (Kosmatka et al. 2002)

Fillers and sealants differ in that fillers are stiffer than sealants and they give support to the edges of the joint. If traffic loads are light, a sealant may be sufficient, but if a joint is subjected to heavier loads, support for joint edges may be needed. Spalling can occur at the edges of saw-cut joints if they are not supported. (Kosmatka et al. 2002) Also, filler is used in expansion joints to form the joint and to allow for room between the joint faces when the concrete expands (ACI 504 1990).

Sealants serve the primary purpose of protecting the concrete by preventing liquids, solids, and gases from passing through the joint. They also keep unwanted material from collecting in the groove of the joint. The sealant must perform these functions while remaining intact and allowing joint movement. Sealants can be divided into two main categories. The first category is field-molded sealants, which are placed while the sealant is in liquid form. The second category is preformed sealants. These sealants are typically preshaped by the manufacturer. Waterstops are a type of preformed sealant that keeps water in or out of the structure. (ACI 504 1990)

Field-molded sealants can be divided into mastics, hot applied, cold applied, chemically curing, and solvent release. Mastics are usually used in structures with small joint movements and where initial costs outweigh the maintenance costs. They do not harden or set after they are applied, but they form a skin layer over the surface. Hot-applied sealants soften when they are heated and harden when they cool. They tend to be inexpensive but they also tend have a shorter life than other sealants. Cold-applied sealants set when they are exposed to a solvent or when emulsions break upon air exposure. These are also typically used in joints that have small movements. Chemically curing sealants are placed while they are in the liquid phase and cure to a hardened state by chemical reactions. They consist of either one- or two-component systems. Also, chemically curing sealants can be used for a wide range of purposes. They resist weathering, are flexible and resilient at hot and cold temperatures, do not readily react with a wide range of chemicals, and have an above-average resistance to abrasion and indentation. In summary, they are considered to be able to withstand greater movements and last longer than other field-molded sealants. Solvent-release sealants cure when a solvent is released. They are similar to cold-applied sealants, but they are affected less by changes in temperature after they have set. Also, solvent-release sealants are typically considered to have an adequate service life. Suitable field-molded joint sealers for water retaining or excluding structures include low melting point asphalt mastic, hot-applied rubber asphalt, hot-applied PVC coal tar, cold-applied rubber asphalt, chemically curing polysulfide, chemically curing polysulfide coal tar, chemically curing polyurethane, and chemically curing silicone. It should be noted that hot-applied rubber asphalts should only be used in horizontal joints because they have been known to fall out of vertical joints when exposed to warm temperatures. (ACI 504 1990)

Preformed sealants can be classified as flexible or rigid waterstops, gaskets and miscellaneous seals, strip (gland) seals, compression seals, impregnated or nonimpregnated flexible foam, and tension-compression seal systems. Rigid waterstops are usually made of steel or copper. Steel waterstops are very stiff and can lead to cracking in the concrete around it. They are typically used in heavy construction projects, such as dams. Copper waterstops are corrosion resistant and are used in dams as well as general construction projects. They also must be handled carefully so that damage does not occur. Because of this and cost issues, flexible waterstops are often used. Flexible waterstops can be made of butyl, neoprene, natural rubbers, and PVC. PVC is the most widely used waterstop material because it can be spliced on site and special configurations can be made for joint intersections. However, rubber waterstops are more elastic. Gaskets are widely used in precast pipe joints. The seal is obtained because the gasket is compressed between the faces of the joint. Strip (gland) seals are typically used in bridge expansion joints. They are basically exposed, flexible waterstops that allow the joint to open or close as needed. Compression seals, as the name entails, must always be in compression. This ensures constant contact between the seal and the joint face even when the joint moves. They perform well in almost all applications and in a wide temperature range. Impregnated flexible foam is a form of compression seal used in buildings and bridges. It does not recover well at cold temperatures, and it does not follow joint movement well. The foam sealant is typically bonded to the faces of the joint. Nonimpregnated flexible foam joint filler resist chemicals well. It can also be custom cut to accommodate any joint size or shape. An adhesive is used to bond the sealant to the faces of the joint. Tension-compression seals are made of molded blocks of elastomeric material. The seal is attached to the faces of the joint and allows movement through grooves and the shear deformation of the elastomeric material. (ACI 504 1990)

If joint sealants are only exposed on one face, the back face of the sealant that is not exposed should not be bonded to the joint. This is so that the sealant can keep the desired shape. A bond breaker is used to achieve this. Polyethylene tape, coated papers, and metal foils can be used as bond breakers. Backup material is also needed in order to keep the sealant from being displaced and to limit the depth of the sealant. Some backup materials do not bond to the sealant and therefore act as a bond breaker. Joint filler can be used as backup material in expansion joints. Neoprene or butyl sponge tubes are used as backup material in large joints, and neoprene or butyl sponge rods are used as backup material in narrow joints (such as contraction joints). Expanded polyethylene, polyurethane, and polyvinylchloride polypropylene flexible foams, metal, plastic, glass fiber, and mineral wool are used as backup material in expansion joints. (ACI 504 1990) Figure 2-24 illustrates how a bond breaker and backup material work.

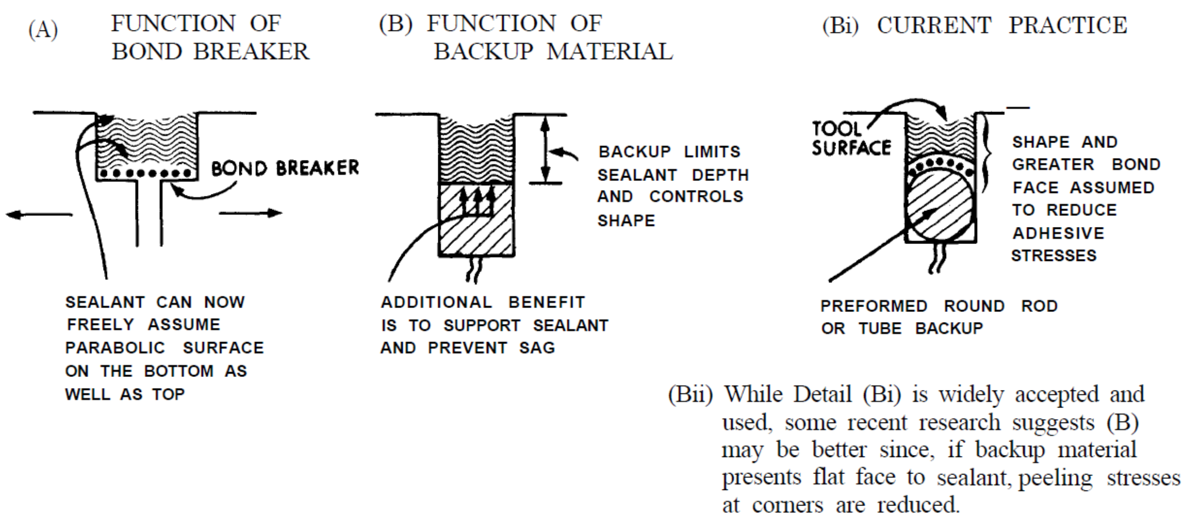


Figure 2-24: How Bond Breakers and Backup Material Work (ACI 504 1990)

2.6 Crack Repair

Before a cracked structure can be repaired, it must be inspected and evaluated to determine the cause and the extent of the cracking. The objective of the crack repair should also be determined. The objectives of crack repair could include restoring or increasing the strength of the structure, restoring or increasing the stiffness of the structure, improving the performance of the structure, making the structure watertight, making the structure aesthetically pleasing, improving the structure's durability, and preventing reinforcement corrosion. (ACI 224 2007)

According to AASHTO (2010b) Section 16.5 cracks, that are a result of thermal effects should not be repaired because the cracks will reopen. However, cracks that are caused by structural movement (e.g. settlement) and that will not open up any wider should be repaired (AASHTO 2010b).

2.6.1 Epoxy Injection

Epoxy injection involves creating closely spaced entry and venting ports along the crack. The exposed surface of the crack is then sealed to prevent the epoxy from leaking before it has hardened. After sealing, the epoxy is injected into the crack with a hydraulic pump, paint pressure pots, or an air-actuated caulking gun. (ACI 224 2007) Figure 2-25 illustrates epoxy resin crack injection.

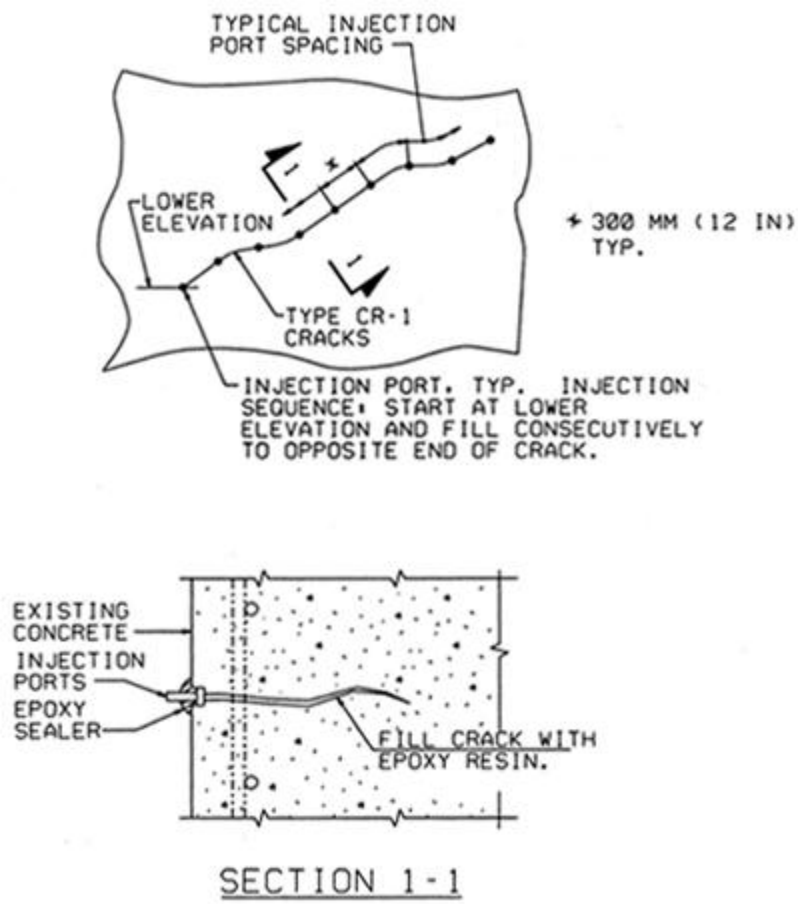


Figure 2-25: Epoxy Resin Crack Injection (FHWA 2005)

Cracks as small as small as 0.005 in. (0.01 mm) can be repaired using epoxy injection. It is typically used to restore the tensile strength of the structural element. Special considerations should be taken if the epoxy will be exposed to high temperatures. Epoxies tend to lose bond strength when they are exposed to high temperatures over a period of time. (ACI 546 2004) It is an effective crack repair method unless the cause of cracking has not been fixed. New cracks are likely to occur next to the original crack in this case. Also, if the crack is continuously leaking, epoxy injection may not be a good option unless a moisture-tolerant epoxy is used. (ACI 224 2007)

In tunnels, cracks can be repaired by injecting epoxy resin into the crack. There are three types of resin used for this application: vinyl ester resin, amine resin, and polyester resin. Vinyl ester resin is generally not usable in tunnels because it will not bond to hardened concrete that is wet. Polyester and amine resins work better for tunnel applications. Their ability to bond to concrete is not affected by the presence of moisture. (AASHTO 2010b)

2.6.2 Routing and Sealing

Routing and sealing of the concrete surface can be used when the crack repair is not a structural repair. It consists of further opening up the crack by sawing or grinding a ¼ in. to 1 in. (6.4 mm to 25 mm) deep groove at the crack location. The groove is cleared of debris and then filled with sealant and allowed to cure. Sealant materials that may be used include epoxies, urethanes, silicones, polysulfides, asphalts, and polymer materials. (ACI 224 2007) An illustration of routing and sealing is shown in Figure 2-26.

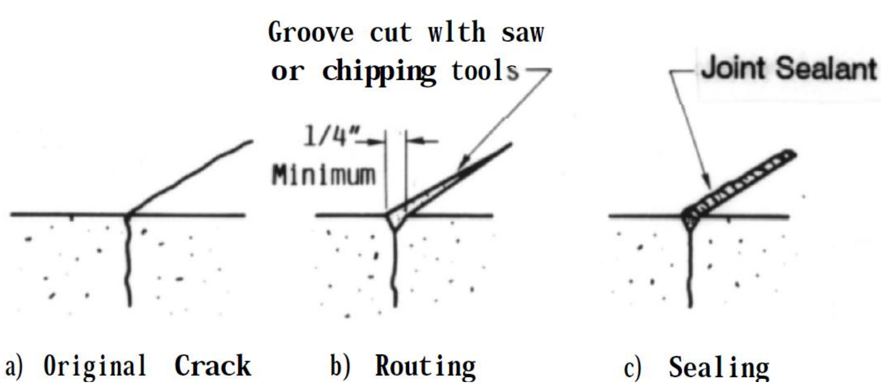


Figure 2-26: Routing and Sealing (Johnson 1965)

Routing and sealing is a method suitable for sealing wide and narrow cracks. Flat horizontal faces are most suitable for routing and sealing, but using it on vertical and round surfaces is also possible. It is also effective for waterproofing surfaces that are subjected to hydraulic pressure or standing water. Active cracks can also be sealed using routing and sealing. A bond breaker between the back of the sealant and the groove must be used in this case to allow the sealant to move with the crack. (ACI 224 2007)

2.7 AASHTO Reinforced Concrete Box Culvert Construction and Design Practice

The requirements for the concrete box culverts as required by the *AASHTO LRFD Bridge Construction Specifications* (2010a) and the *AASHTO LRFD Bridge Design Specifications* (2007) are reviewed in this section.

2.7.1 Wall Height Construction Limitations

Section 8.7.2.4 (AASHTO 2010a) states that the bottom slab or footing should be the first part of the CIP box culvert placed, and it should be allowed to set before the walls and the top slab are placed. If the heights of the walls are 5 ft (2 m) or shorter, the top slab and the walls can be placed as a monolith (AASHTO 2010a).

Section 8.7.2.1 (AASHTO 2010a) states that if the culvert walls are taller than 15 ft (4.6 m), they should be permitted to set for at least 12 hours before casting the top slab. If the culvert walls are 15 ft (4.6 m) high or smaller but greater than 5 ft (2 m) high, the walls should be allowed to set for at least 30 minutes. These setting times are to allow for the concrete to settle after losing water due to bleeding. (AASHTO 2010a)

2.7.2 Concrete Protection from Environmental Conditions

Section 8.6.1 (AASHTO 2010a) states that the concrete temperature must be between 50 and 90 ^{o}F (10 and 32 ^{o}C) prior to it being placed.

2.7.2.1 Protection from Hot-Weather Conditions

Section 8.6.3 (AASHTO 2010a) states that if the ambient temperature is greater than 90 ^{o}F (32.2 ^{o}C), objects that the fresh concrete will be placed against (forms, reinforcement, etc.) must be cooled to a temperature that is less than 90 ^{o}F (32 ^{o}C). This can be done by spraying the items with water. The temperature of the concrete must be controlled also. This can be done by keeping the concrete materials and the mixing equipment in the shade, splashing the aggregate with water, keeping the aggregate in a refrigerated area, replacing a portion of or all of the mixing water with crushed ice, or by adding liquid nitrogen to the concrete mix. (AASHTO 2010a)

2.7.2.2 Protection from Cold-Weather Conditions

Section 8.6.4 (AASHTO 2010a) states that if the temperature of the air is lower than 35 ^{o}F (2 ^{o}C), the concrete should be kept at a temperature greater than or equal to 45 ^{o}F (7 ^{o}C) for the first 6 days after it has been placed. If supplementary cementing materials (SCM) are used, longer periods of time may be required unless 65 percent of the concrete's specified compressive strength has been reached at 6 days. When the concrete is being placed, the concrete temperature shall be at least 60 ^{o}F (16 ^{o}C) for members that are smaller than 12 in. (300 mm) thick. To get concrete that is within the specified temperature range, the aggregate or the mix water may be heated. (AASHTO 2010a)

2.7.3 Joints

2.7.3.1 Construction Joints

Section 8.8.1 (AASHTO 2010a) requires that construction joints, unless otherwise approved, only be put at places where indicated on the plans, or where the concrete placement schedule calls for them. All planned reinforcement should be continuous through the joint at these locations (AASHTO 2010a).

For bonding purposes, Section 8.8.2 (AASHTO 2010a) requires that vertical joints must be constructed with keys, but horizontal construction joints may be constructed with or without keys. The keys should be formed by making depressions in the concrete that take up approximately one-third of the contact surface (AASHTO 2010a).

2.7.3.2 Contraction Joints

Section 8.9.1 (AASHTO 2010a) requires that contraction joints be put at places indicated on the construction plans and should be constructed as specified. The joints may be open joints, filled joints, sealed joints, reinforced joints, or joints with a combination of these features (AASHTO 2010 a).

Section 11.6.1.5.2 (AASHTO 2007) requires that contraction joints be placed at a spacing of no more than 30 ft (9.1 m) in wingwalls.

Section 8.9.2.3 (AASHTO 2010a) states that materials put inside of a contraction joint should be a bond breaking material such as asphalt-saturated felt paper. When joint sealant is required in contraction joints, Section 8.9.2.4 (AASHTO 2010a) requires that the sealants used must be either a hot-poured sealant that conforms to AASHTO M 282 (ASTM D3406), a silicon cold-poured sealant that follows Federal Specification TT-S-1543 Class A, or an impervious, commercial quality polyethylene foam strip.

2.7.3.3 Expansion Joints

Section 8.9.1 (AASHTO 2010a) requires expansion joints be placed at locations indicated on the construction plans and be constructed as specified. The joints may be open joints, filled joints, sealed joints, reinforced joints, or joints with a combination of these features (AASHTO 2010a).

Section 11.6.1.5.2 (AASHTO 2007) requires that expansion joints be placed at a spacing of no more than 90 ft (27 m) in wingwalls.

When joint filler is required in expansion joints, Section 8.9.2.1 (AASHTO 2010a) requires that the preformed joint fillers for concrete pavements and structural construction should conform to AASHTO M 213 (ASTM D1751), preformed sponge rubber and cork joint fillers should conform to AASHTO M 153 (ASTM D1752), and preformed joint fillers for concrete should conform to AASHTO M 33 (ASTM D994). Section 8.9.2.2 (AASHTO 2010a) states that polystyrene board filler made of expanded polystyrene may also be used.

When joint sealant is required in expansion joints, Section 8.9.2.4 (AASHTO 2010a) requires that the sealants used must be either a hot-poured sealant that conforms to AASHTO M 282 (ASTM D3406), a silicon cold-poured sealant that conforms to Federal Specification TT-S-1543 Class A, or an impervious, commercial quality polyethylene foam strip.

2.7.3.4 Waterstops

Section 8.9.2.6 (AASHTO 2010a) requires waterstops be made of polyvinyl chloride (PVC), copper or rubber. They should also be uniform throughout, dense, and without imperfections (AASHTO 2010a).

Also, Section 8.9.3.4 (AASHTO 2010a) requires that when waterstops are placed at a joint that is free to move (i.e. contraction joint), they shall allow the joint to move without being damaged themselves. They shall be placed in the joint so that a continuous waterproof seal is formed (AASHTO 2010a).

2.7.3.5 Concrete Culvert Joint Sealants

Section 27.4.2 (AASHTO 2010a) requires culvert joints be sealed so as to keep outside water and soil from entering the culvert. Section 27.3.3 (AASHTO 2010a) states that the sealant can be made of cement mortar, flexible watertight gaskets, conforming to AASHTO M 198

(ASTM C990) or AASHTO M 315 (ASTM C433), or other materials that are approved by the engineer.

2.7.4 Reinforcement

Section 5.10.8 of the *AASHTO LRFD Bridge Design Specifications* (2007) states that shrinkage and temperature reinforcement in CIP reinforced concrete box culverts components should be governed by the two following equations:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y} \qquad \textbf{Equation 2-3}$$

where,

A_s = area of temperature and shrinkage reinforcement per length of culvert component (in.^{2}/ft),

b = smallest height/width of the culvert component section (in.),

h = smallest thickness of the culvert component section (in.), and

f_y = yield strength of the reinforcement $\leq$ 75 ksi (ksi).

$$0.11 \text{ in.}^2/\text{ft} \leq A_s \leq 0.60 \text{ in.}^2/\text{ft} \qquad \textbf{Equation 2-4}$$

The temperature and shrinkage reinforcement from these equations should be distributed uniformly around the perimeter of the culvert component. Equation 2-3 was derived so that the temperature and shrinkage reinforcement ratio would be 0.0018 for Grade 60 reinforcement, the value specified by ACI 318 (2011). (AASHTO 2007)

For a culvert wall with a height of 8 ft (2 m), a thickness of 12 in. (305 mm), and an assumed reinforcement yield strength of 60 ksi (414 MPa), the temperature and shrinkage reinforcement area would be 0.116 in.^{2}/ft (245 mm^2/m). If the same wall was 30 in. (762 mm) thick, the temperature and shrinkage reinforcement area would be 0.248 in.^{2}/ft (525 mm^2/m).

2.8 Box Culvert Construction Practices of SASHTO States

States that are members of the Southeastern Association of State Highway and Transportation Officials (SASHTO) have climates that are similar to Alabama. Therefore these states may have similarities in requirements in requirements for reinforced concrete. CIP, box culvert design and construction methods when compared to ALDOT. Puerto Rico was excluded because it is not a U.S. State.

Information was difficult to find for most states; therefore, some states may have significantly less information than others.

2.8.1 Arkansas State Highway and Transportation Department (AHTD)

2.8.1.1 General Construction

The AHTD uses both precast and CIP reinforced concrete box culverts. When precast box culverts are used, they must conform to AASHTO M 259 or M 273. (AHTD 2003)

2.8.1.2 Wall Height Construction Limitations

For CIP box culverts that are 6 ft (2 m) in height or smaller, the top slab and walls can be built monolithically. If they are built in separate sections, the top slab will be bonded to the walls, and the walls will be bonded to the bottom slab by roughened longitudinal keys. Also, the top slab and walls should be cast 24 hours or more after the previous placement has set. (AHTD 2003)

2.8.1.3 Concrete Protection from Environmental Conditions

2.8.1.3.1 Protection from Hot-Weather Conditions

The temperature of the concrete mix when it is placed should not be greater than 95 ^{o}F (35 ^{o}C) (AHTD 2003).

2.8.1.3.2 Protection from Cold-Weather Conditions

Concrete should not be placed when the air temperature is below 36 ^{o}F (2 ^{o}C). An exception is when the concrete will be enclosed, and the mix ingredients and the enclosed space will be heated. Concrete should not be placed unless the concrete mix temperature is 50 ^{o}F (10 ^{o}C) or above. (AHTD 2003)

2.8.1.4 Joints

2.8.1.4.1 Joint Waterproofing

Waterstops should be made of copper, rubber, or polyvinyl chloride (PVC) and they should be uniform throughout, dense, and have no porosity (AHTD 2003).

AHTD Standard Drawing RCB-1 (2006) states that construction joints in the side walls and top slab of box culverts should have membrane waterproofing (AHTD 2006).

2.8.1.4.2 Joint Filler

Expansion joint filler can be preformed sponge rubber complying with AASHTO M 153 Type I or a mixture of one part asphalt and four parts sawdust (AHTD 2003).

2.8.1.4.3 Joint Sealant

Sealants that can be used for contraction joints include cold-applied silicone, hot-poured elastomeric joint sealant, synthetic polymer joint sealant, or hot-poured elastic type sealant. A backer rod can be used with the cold-applied silicone sealants that either need a primer to bond with concrete or that do not need one. It can also be used with hot-poured elastomeric joint sealants. (AHTD 2003)

2.8.2 Florida Department of Transportation (FDOT)

2.8.2.1 General Construction

The FDOT uses both CIP and precast box culverts. It is the contractor's choice unless there is a plan note forbidding the use of precast culverts. (FDOT 2010c) A typical double barrel CIP box culvert cross section used by the FDOT is shown in Figure 2-26.

For CIP culverts, the base slab (or footing) should be placed first and allowed to set before construction is continued. The bottom slab, footing, and apron walls should be built as a monolith if possible. (FDOT 2010c)

The wingwalls are recommended to be placed as monoliths. When this is not possible, the construction joints should be horizontal and not visible. (FDOT 2010c) A typical double barrel culvert cross section used by the FDOT is shown in Figure 2-27.

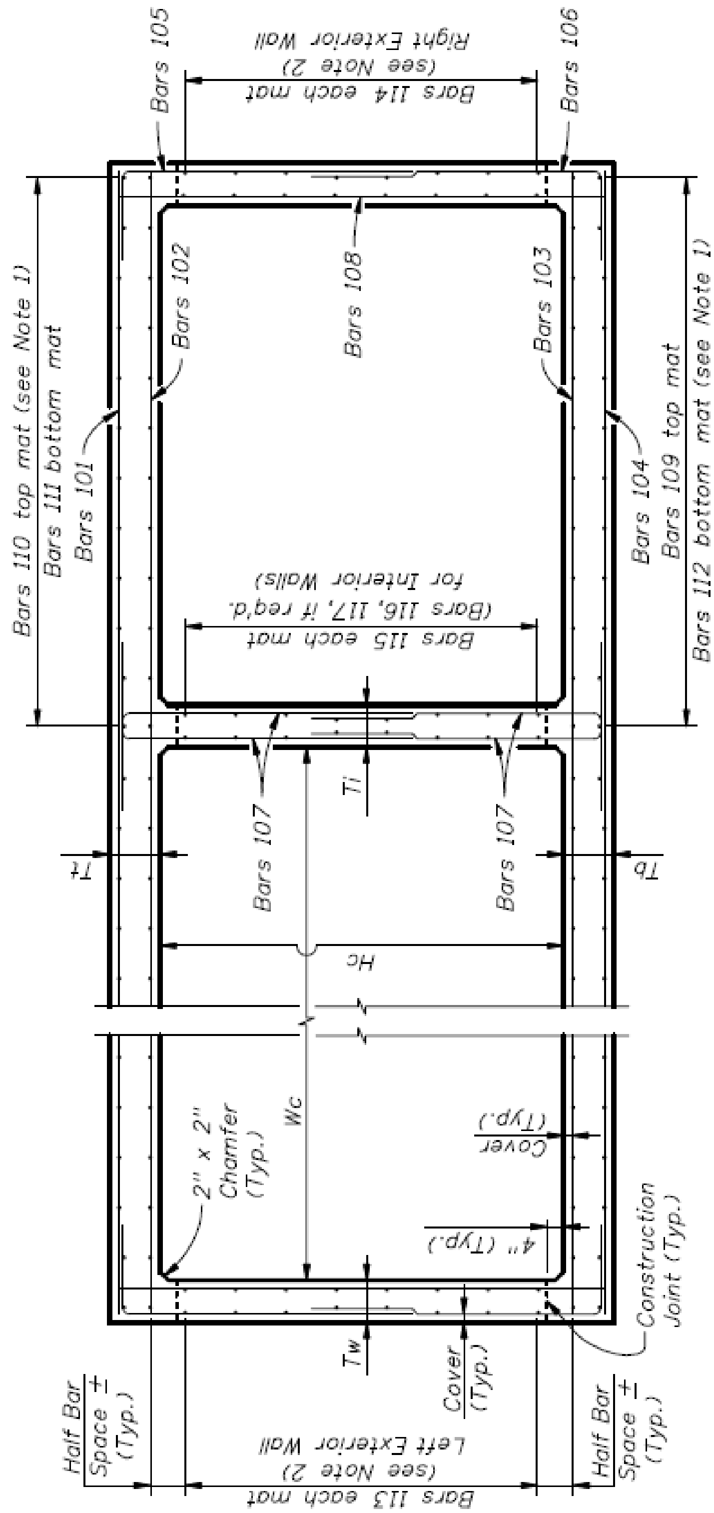


Figure 2-27: Typical FDOT Double Barrel Cast-in-Place Box Culvert Cross Section (FDOT 2010a)

2.8.2.2 Wall Height Construction Limitations

If the walls of the culvert are 6 ft (2 m) or shorter, the top slab and side walls may be poured as a monolith. It is also allowable for the side walls to be poured first, and then have the top slab poured after the walls have set. If the walls of the culvert are taller than 6 ft (2 m), the sidewalls should be placed first and allowed to set for twelve hours before placing the top slab. (FDOT 2010c)

The smallest allowable CIP box culvert is 4 ft by 4ft (1 m by 1 m), and the smallest allowable precast box culvert is 3 ft by 3 ft (0.9 m by 0.9 m) (FDOT 2010b).

2.8.2.3 Concrete Protection from Environmental Conditions

2.8.2.3.1 Protection from Hot-Weather Conditions

The FDOT *Standard Specifications for Road and Bridge Construction* (2010c) defines hot weather concreting as "the production, placing and curing of concrete when the concrete temperature at placing exceeds 85 ^{o}F (29 ^{o}C) but is less than 100 ^{o}F (37 ^{o}C)." All concrete that has a temperature greater than 100 ^{o}F (37 ^{o}C) should be refused. If hot weather concrete measures are not in effect, all concrete with a temperature greater than 85 ^{o}F (29 ^{o}C) should be refused. If the temperature of the placed concrete is greater than 75 ^{o}F (23 ^{o}C), a water reducing admixture or a water-reducing and retarding admixture should be added to the concrete mix. Also, forms and reinforcement shall be sprayed with cold water before concrete is placed in hot weather conditions. (FDOT 2010c)

2.8.2.3.2 Protection from Cold-Weather Conditions

No concrete should be placed when the temperature of the concrete is below 45 ^{o}F (7 ^{o}C). Also, concrete should not be mixed when the temperature of the air is below 45 ^{o}F (7 ^{o}C) and dropping. However, concrete may be mixed and placed if the air temperature reading taken is in the shade and is at least 40 ^{o}F (4 ^{o}C) and rising. If the fresh concrete is not heat cured, it should be protected from freezing until its compressive strength is at least 1,500 psi (10 MPa). (FDOT 2010c)

2.8.2.4 Joints

2.8.2.4.1 Construction Joints

When building the walls and the top slab as a monolith, any essential construction joints should be vertical and have beveled keys. Transverse construction joints should be perpendicular to the culvert barrel. Proper provisions for longitudinal and transverse keys at joints should be made. The keys should be beveled and not more than 1.5 in. (38 mm) from the edge of the concrete. Also, transverse construction joints should be vertical and have continuous reinforcement through them. In long concrete box culverts, vertical construction joints have a minimum spacing of 30 ft (9.1 m). (FDOT 2010c)

2.8.2.4.2 Joint Waterproofing

When a dry environment is desired in the culvert, an external sealing band (in compliance with ASTM C87) should be used to waterproof the culvert joints. It should be centered on the joint and be placed from the bottom of one sidewall to the top slab, on the top slab, and then from the top slab to the bottom of the other sidewall. The band should be placed at individual precast section joints or at CIP construction joints. (FDOT 2011)

2.8.2.4.3 Joint Filler

Joint fillers should meet AASHTO M-153 or AASHTO M-213. If the filler is made of cellulose fiber it should meet AASHTO M-213 requirements (excluding the asphalt content requirement) if it contains at least 0.2% zinc borate and 1.5% waterproofing wax. Type I, Type II, or Type III AASHTO M-153 joint sealers may be used unless otherwise specified. (FDOT 2010c)

2.8.2.4.4 Joint Sealant

Joint sealants can be made of many different materials, but they will typically be a mixture of bituminous based materials. The material will melt when it is heated and then adhere to the concrete to form a seal when it cools. A low modulus silicone sealant may also be used as a joint sealant. Acetic acid cure sealants should not be used. (FDOT 2010c)

A bond breaker rod made of expanded polyethylene foam may be used. The rod should not react with or bond to the sealant. (FDOT 2010c)

2.8.3 Georgia Department of Transportation (GDOT)

2.8.3.1 General Construction

The GDOT allows for the use of precast and CIP reinforced concrete box culverts (GDOT 1985). GDOT also recommends that one foot be the smallest fill height allowed for a reinforced concrete box culvert (GDOT 2010). An example of a single barrel CIP box culvert cross section used by the GDOT is shown in Figure 2-28.

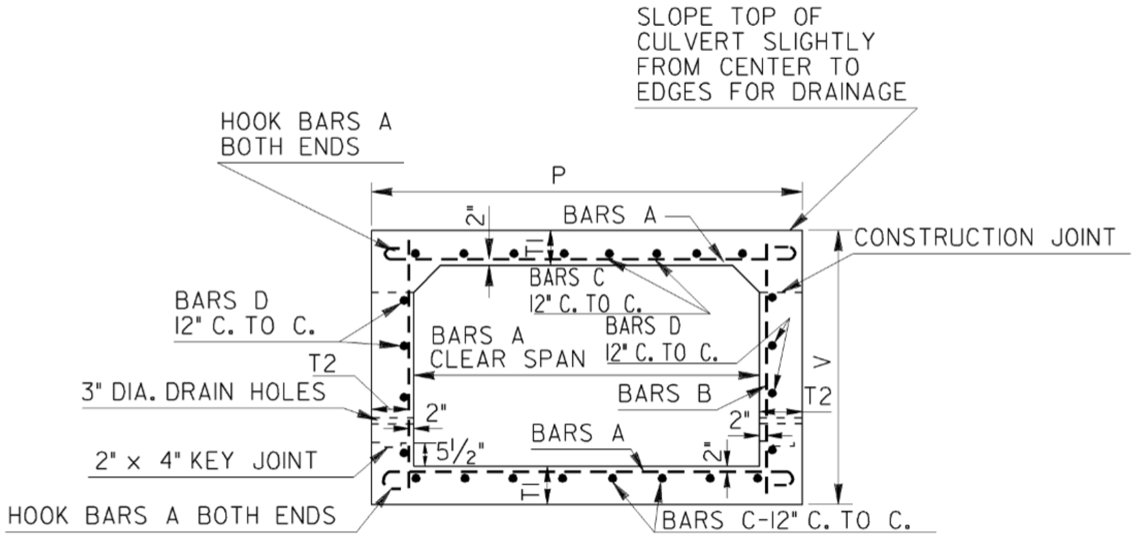


Figure 2-28: GDOT Single Barrel Cast-in-Place Box Culvert Cross Section Example (GDOT 2001a)

2.8.3.2 Concrete Protection from Environmental Conditions

2.8.3.2.1 Protection from Cold-Weather Conditions

The concrete temperature should be kept above 50 ^{o}F (10 ^{o}C) for 72 hours after it has been placed, and it also should be kept above freezing temperature for 6 days after it has been placed. This can be achieved by using heated enclosures, commercial blankets, or batt insulation if the expected 48-hour temperature is below 25 ^{o}F (-4 ^{o}C). If the expected 48 hour temperature is at or above 25 ^{o}F (-4 ^{o}C), heavy-duty polyethylene sheets can be used. (GDOT 2001b)

2.8.3.3 Joints

2.8.3.3.1 Construction Joints

Transverse construction joints should be perpendicular to the culvert barrel. They should be at all locations where the design changes and at places shown on the plans. Construction joints should not be located in the culvert section directly under the width of the pavement. However, if construction joints must be presented directly under the pavement width then the reinforcement should be continuous through the joint and no bond breaking measures should be taken. Reinforcement should also be continuous when the joint is within 15 ft (4.6 m) of the end of the culvert or when it is located directly under a roadway. (GDOT 1996) All other construction joints should not have continuous reinforcement through the joint. The maximum joint spacing is 30 ft (9.1 m) (GDOT 2010).

2.8.3.3.2 Joint Waterproofing

Coal-tar pitch is used for waterproofing in culverts (GDOT 2001b).

2.8.3.3.3 Joint Filler

Preformed joint filler, preformed foam joint filler, elastomeric polymer type joint compound, or water-blown urethane can be used as joint filler. Preformed joint filler should meet AASHTO M 153 or AASHTO M 213 specifications. (GDOT 2001b)

2.8.3.3.4 Joint Sealant

Hot-poured joint sealer, preformed elastic joint sealer, or silicone sealant with a bond breaker can all be used as joint sealants. A backer rod should be used for the bond breaker. (GDOT 2001b)

2.8.4 Kentucky Transportation Cabinet (KYTC)

2.8.4.1 General Construction

The KYTC recommends that a minimum fill height of 1 ft (0.3 m) should be for culverts used unless the top slab is designed to be the driving surface (KYTC 2005). The KYTC also allows the use of precast or CIP reinforced concrete box culverts (KYTC 2008).

The base slab and footings of a box culvert should be poured and allowed to cure before placing the walls and top slab. Also, the base slab and footings should be poured as a monolith when it is possible. When necessary, construction joints should be perpendicular to the culvert barrel. (KYTC 2008)

Wingwalls should be poured as a monolith if possible. However, if that is not possible, horizontal or vertical joints may be used. (KYTC 2008)

The top slab should have a minimum thickness of 7 in. (178 mm). The bottom slab should have the same effective depth as the top slab but it should have a total depth that is 1 in. (25 mm) greater for single barrel culverts and 2 in. (51 mm) greater for multiple barrel culverts. Culvert sidewalls should have a minimum thickness of 1/12th of the clear height or 10 in. (254 mm). Interior culvert wall should have a minimum thickness of 10 in. (254 mm). (KYTC 2005)

A typical CIP cross section of a single barrel culvert used by the KYTC from Exhibit 510 of the KYTC *Division of Structural Design Guidance Manual* (2005) is shown in Figure 2-29.

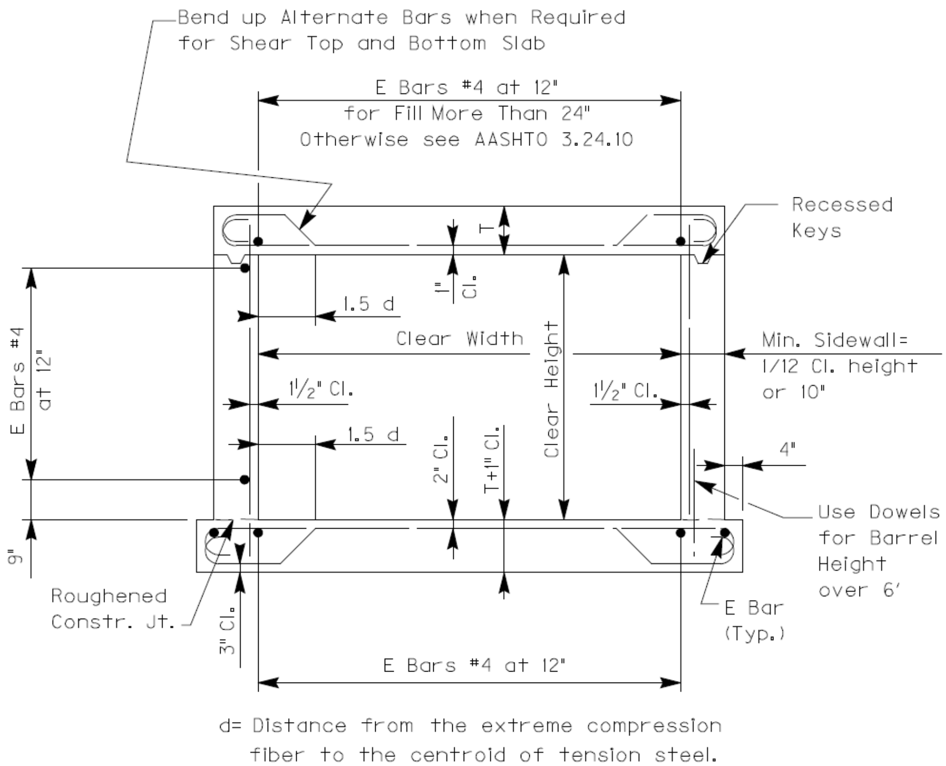


Figure 2-29: Typical KYTC Single Barrel Cast-in-Place Box Culvert Cross Section (KYTC 2005)

2.8.4.2 Wall Height Construction Limitations

If the clear height of a culvert is less than 5 ft (2 m), the walls and top slab of a box culvert can be placed as a monolith if it is desired. The construction joints in this case should be vertical and perpendicular to the axis of the culvert. If the clear height is 5 ft (2 m) or greater, the side walls should be placed before the top slab. (KYTC 2008)

The minimum size for a CIP concrete box culvert is 4 ft x 4 ft (1 m x 1 m) (height x span). The maximum size is 16 ft x 20 ft (4.9 m x 6.1 m). (KYTC 2011)

2.8.4.3 Concrete Protection from Environmental Conditions

The concrete temperature just before a box culvert is placed should be between 50 and 90 ^{o}F (10 and 32 ^{o}C) (KYTC 2008).

2.8.4.3.1 Protection from Hot-Weather Conditions

If the ambient air temperature is above 90 ^{o}F (32 ^{o}C), all surfaces that will touch the placed concrete should be cooled to below 90 ^{o}F (32 ^{o}C). If the ambient air temperature is greater than 100 ^{o}F (38 ^{o}C), concrete for box culverts should not be placed. (KYTC 2008)

2.8.4.3.2 Protection from Cold-Weather Conditions

Freshly placed concrete should be kept at a temperature no lower than 45 ^{o}F (7 ^{o}C) for the first 3 days after it has been placed, and it should be kept at a temperature no lower than 40 ^{o}F (4 ^{o}C) for another 4-days after that. Concrete should not be placed when temperatures are expected to drop below these limits unless measures have been taken to sustain acceptable concrete temperatures. In order to keep the concrete at an acceptable temperature, the water and aggregates should be heated. Also, freshly placed concrete should not come into contact with objects (forms, etc.) covered with frost or that are at a temperature at or below 32 ^{o}F (0 ^{o}C). (KYTC 2008)

2.8.4.4 Joints

2.8.4.4.1 Construction Joints

Wood strips that have been saturated should be used to form keys in construction joints. Steel dowels may also be used as an alternative to shear keys. (KYTC 2008) Construction joints between the walls and top slab should have keys. The keys should be turned down, as shown in Figure 2-31. (KYTC 2005)

2.8.4.4.2 Contraction Joints

Contraction joints can be used in culvert wingwalls if it is approved by the division of structural design (KYTC 2005).

A contraction joint detail for walls, which appears applicable to culverts, from Exhibit 516 of the KYTC *Division of Structural Design Guidance Manual* (2005) is shown in Figure 2-30.

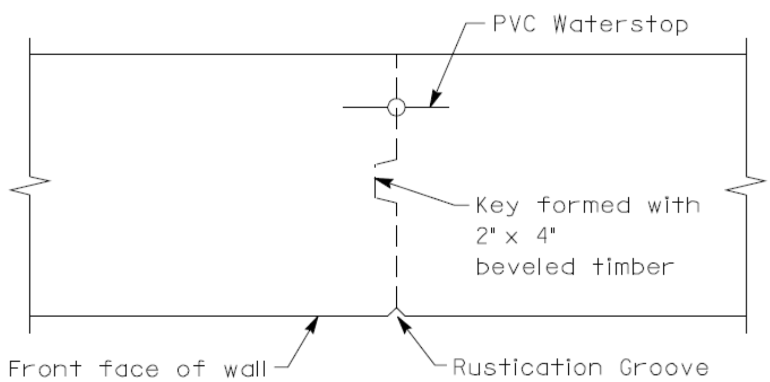


Figure 2-30: KYTC Wall Contraction Joint Detail (KYTC 2005)

2.8.4.4.3 Expansion Joints

Expansion joints can be used in culvert wingwalls if it is approved by the division of structural design (KYTC 2005).

An expansion joint detail for walls, which appears applicable to culverts, from Exhibit 516 of the KYTC *Division of Structural Design Guidance Manual* (2005) is shown in Figure 2-31.

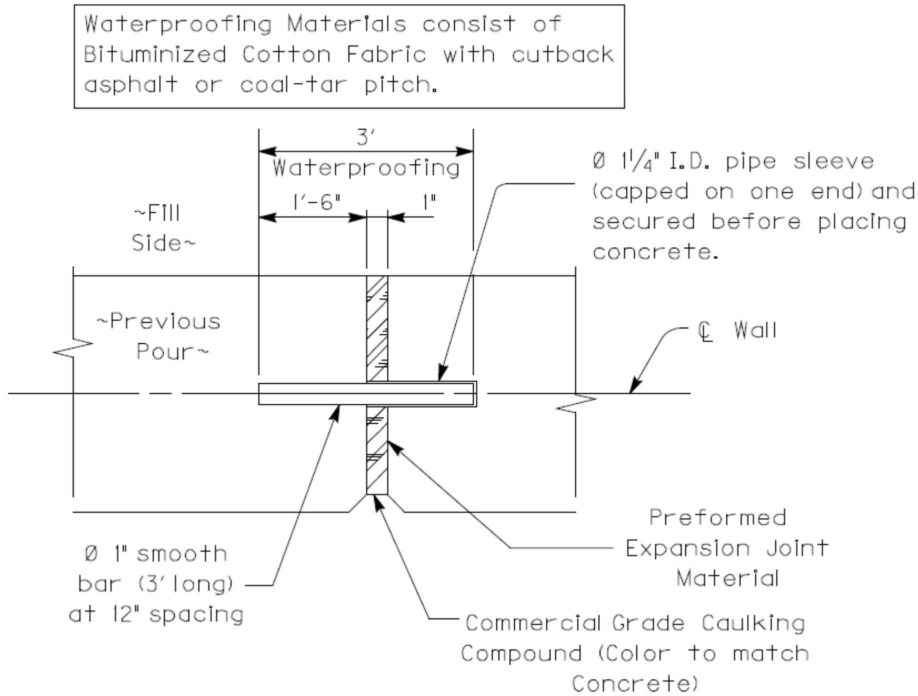


Figure 2-31: KYTC Wall Expansion Joint Detail (KYTC 2005)

2.8.4.4.4 Joint Filler

Preformed filler should be made of a single piece unless otherwise specified. Preformed sponge rubber, preformed cork, or preformed asphalt may be used for expansion joints. (KYTC 2008)

2.8.4.4.5 Joint Sealant

Hot-poured elastic sealant, silicone rubber, and preformed expansion and compression joint sealers can be used as joint sealants. The hot-poured sealant should meet ASTM D 6690 Type II requirements, the preformed expansion joint sealant should meet ASTM D 5973, and the preformed compression joint sealant should meet ASTM D 2628. (KYTC 2008)

2.8.5 Lousiana Department of Transportation and Development (La DOTD)

2.8.5.1 General Construction

The La DOTD allows the use of CIP or precast reinforced concrete box culverts. When using CIP culverts, the footings or base slab should be placed and allowed to set before proceeding further with construction. (La DOTD 2006)

Wingwalls should be built as a monolith when possible. Otherwise, horizontal construction joints that are not visible should be used. (La DOTD 2006)

Figure 2-32 shows the cross section of a typical La DOTD double barrel CIP box culvert.

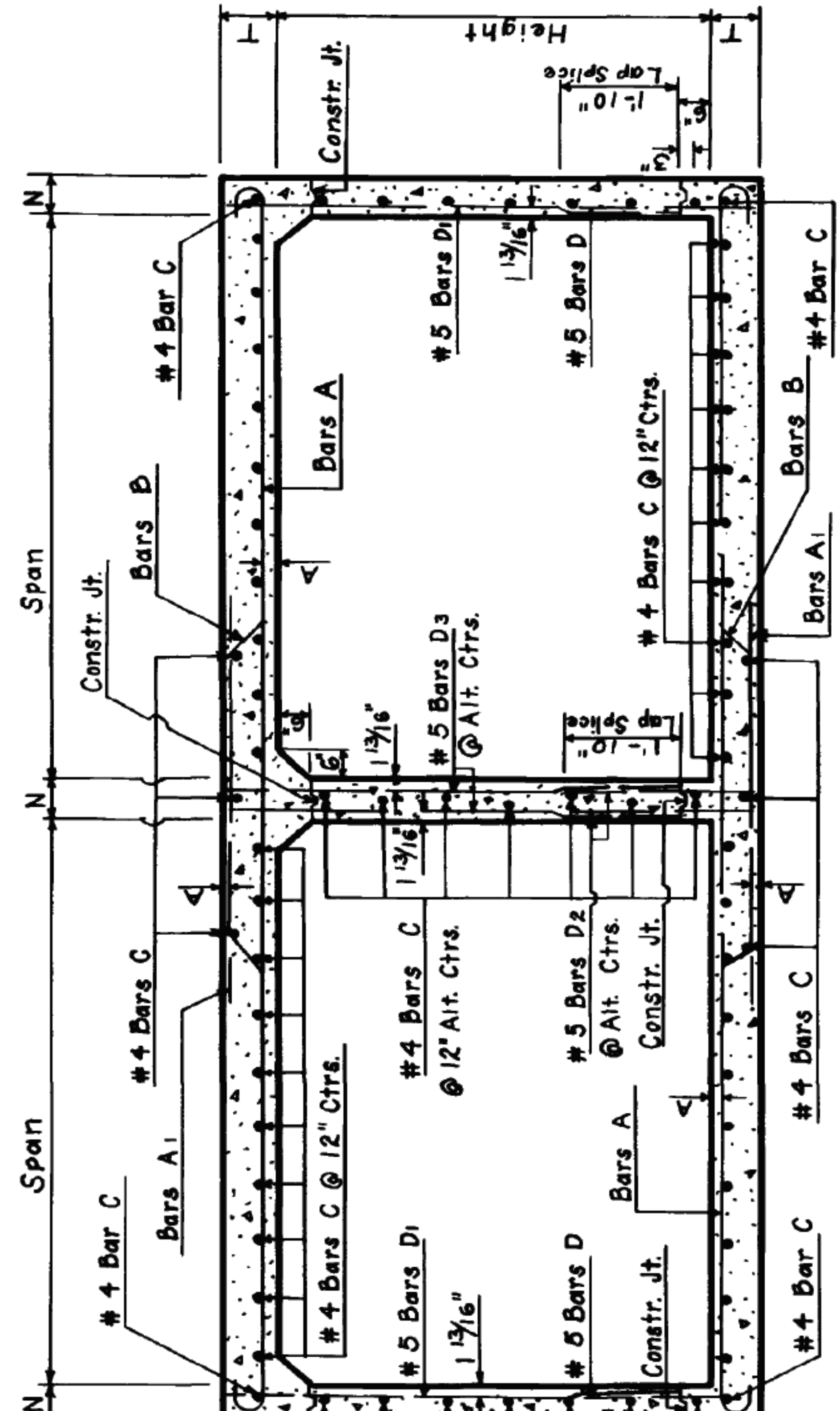


Figure 2-32: Typical La DOTD Double Barrel Cast-in-Place Box Culvert Cross Section (La DOTD 1978)

2.8.5.2 Wall Height Construction Limitations

When a culvert is 4 ft (1 m) high or less, the top slab and walls can be built as a monolith. When this is done, the construction joints should be normal to the axis of the culvert and vertical. When a culvert is larger than 4 ft (1 m) high, the walls shall be placed first and allowed to cure before the top slab can be placed. (La DOTD 2006) The location of construction joints in these cases is shown in Figure 2-34.

The minimum size for a reinforced concrete box culvert is 4 ft by 4 ft (1 m by 1 m) (La DOTD 1987).

2.8.5.3 Concrete Protection from Environmental Conditions

2.8.5.3.1 Protection from Cold-Weather Conditions

When air temperature in the shade is dropping and it reaches 40 ^{o}F (4 ^{o}C), all concrete operations should be stopped. They can resume when the air temperature in the shade is rising, is at least 35 ^{o}F (2 ^{o}C), and the forecasted high temperature is above 40 ^{o}F (4 ^{o}C). (La DOTD 2006)

The aggregate can be heated to allow for the concrete to be placed when the temperature is less than 35 ^{o}F (2 ^{o}C) and the procedure is approved in writing. The placed concrete must then be protected using insulating materials, additional covering, or by other approved methods. (La DOTD 2006)

2.8.5.4 Joints

The locations of longitudinal construction joints and the wingwall expansion joint for La DOTD box culverts are shown in Figure 2-33.

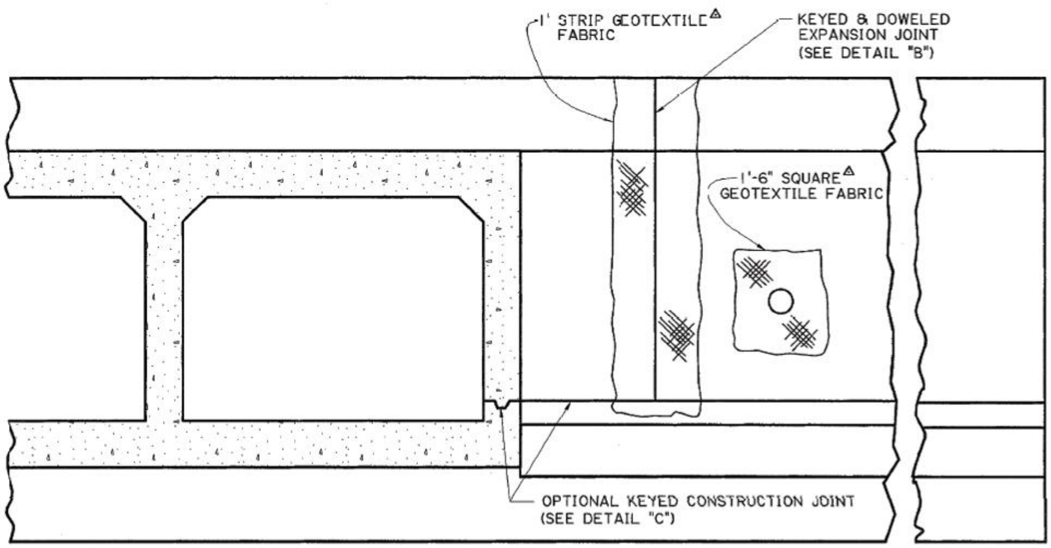


Figure 2-33: La DOTD Box Culvert Section Showing the Rear Side of a Wingwall and Expansion and Construction Joint Locations (La DOTD 2008)

2.8.5.4.1 Construction Joints

The longitudinal construction joints between walls and base slabs should be keyed and located at points shown in Figure 2-33 (La DOTD 2008). A detail of the La DOTD construction joint is shown in Figure 2-34.

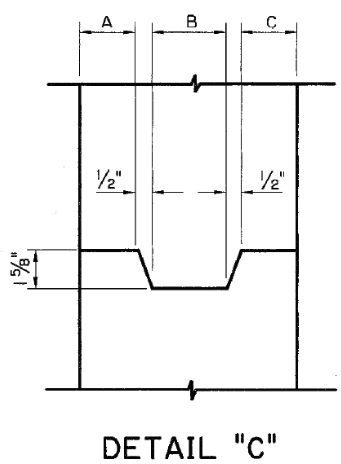


Figure 2-34: La DOTD Longitudinal Construction Joint between Base and Wall (La DOTD 2008)

2.8.5.4.2 Expansion Joints

Transverse expansion joints in culvert barrels should be spaced at 200 ft (61.0 m) and should be keyed. No reinforcement should run through these joints. The expansion joint filler material should be a preformed resilient bituminous type, and geotextile should be placed around the outside of the culvert barrel at the expansion joint. (La DOTD 2008) The location of the transverse expansion joint is shown in Figure 2-35. Note the concrete block located under the base of the culvert at the expansion joint. A typical transverse expansion joint used in culvert barrels by the La DOTD is shown in Figure 2-36.

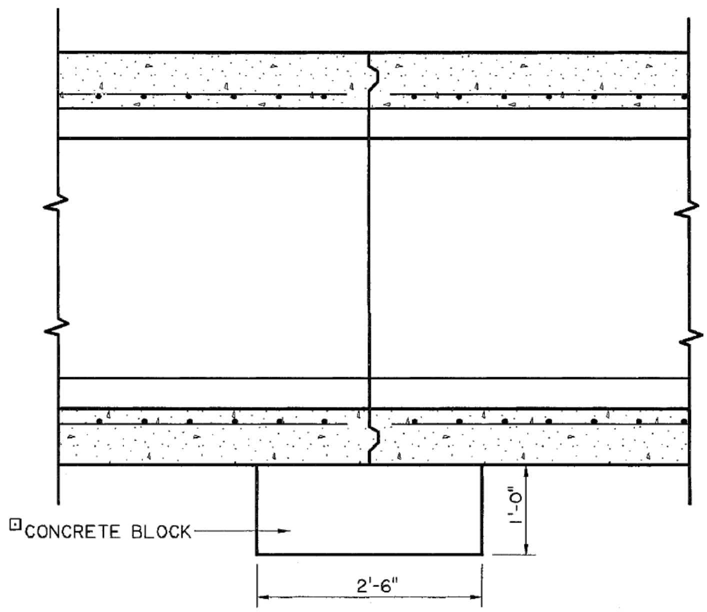


Figure 2-35: La DOTD Culvert Barrel Elevation View and Transverse Expansion Joint Location (La DOTD 2008)

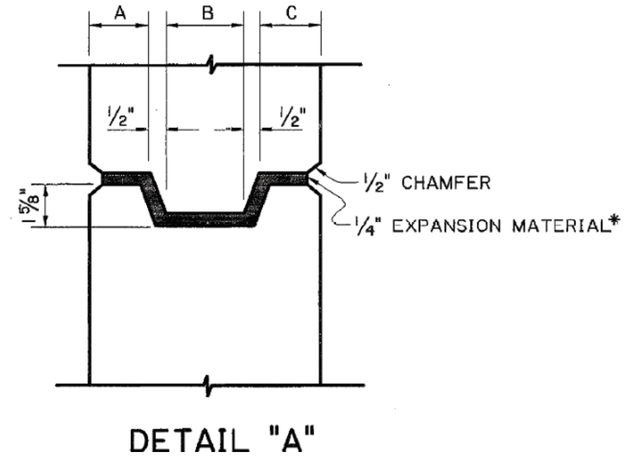


Figure: 2-36: La DOTD Transverse Expansion Joint Details for Culvert Barrels (La DOTD 2008)

Expansion joints in wingwalls should be keyed vertical joints and should be dowelled. The expansion joint filler material should be preformed resilient bituminous type. A 1-foot (0.3 m) wide strip of geotextile fabric should be put on the back side of the wingwall expansion joint, as shown in Figure 2-33. (La DOTD 2008) The expansion joint should be located as shown in Figure 2-33. A detail of a wingwall expansion joint is shown in Figure 2-37.

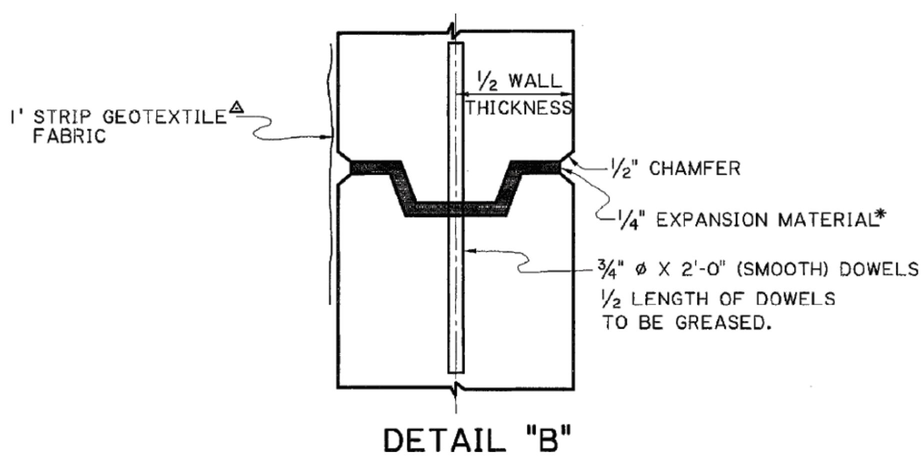


Figure 2-37: La DOTD Wingwall Expansion Joint Detail (La DOTD 2008)

2.8.5.4.3 Joint Waterproofing

Waterstops can be made of plastic, rubber, or metal. They should also allow for the joint to move without being damaged themselves. (La DOTD 2006)

2.8.5.4.4 Joint Filler

Joint fillers can be preformed. They can be bituminous, wood, asphalt ribbon, closed cell polyethylene, or rubber. (La DOTD 2006)

2.8.5.4.5 Joint Sealant

Joint sealers can be hot-poured rubberized asphaltic type, polyurethane, or silicone. A backer material should be used along with the sealant. (La DOTD 2006)

2.8.6 Mississippi Department of Transportation (MDOT)

The minimum size for a reinforced concrete box culvert is 4 ft by 4 ft (1 m by 1 m) (MDOT 2011).

2.8.7 North Carolina Department of Transportation (NCDOT)

2.8.7.1 General Construction

The NCDOT uses CIP reinforced concrete box culverts with an option of using precast culverts. Precast box culverts are not allowed if the maximum design fill on top of the culvert is exceeded (10 ft [3.0 m] in some areas of North Carolina and 15 ft [4.6 m] in others). (NCDOT 2007)

An example of a CIP box culvert cross section is shown in Figure 2-38, which is from Figure 9-4 of the NCDOT *Structure Design Unit Design Manual* (2007).

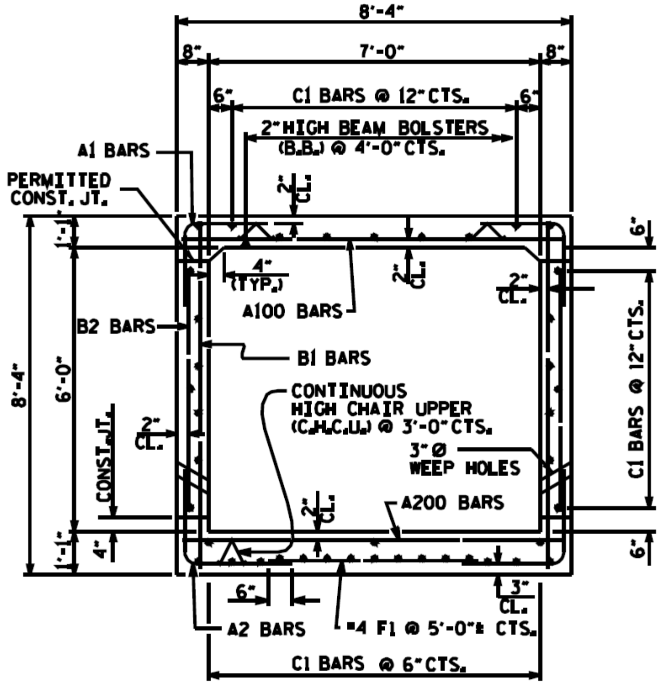


Figure 2-38: NCDOT Single Barrel Cast-in-Place Box Culvert Cross Section Example (NCDOT 2007)

2.8.7.2 Wall Height Construction Limitations

Culverts with a vertical clearance of 4 ft (1 m) or smaller must be poured as monoliths. Culverts with a vertical clearance of 4 ft (1 m) through 8 ft (2 m) can have longitudinal construction joints below fillets in the top slab, but it is not required. Culverts with a vertical clearance of 9 ft (3 m) or larger must have longitudinal construction joints below the fillets in the top slab. (NCDOT 2007)

2.8.7.3 Concrete Protection from Environmental Conditions

The temperature of the concrete should be no less than 50 ^{o}F (10 ^{o}C) and no higher than 95 ^{o}F (35 ^{o}C) when it is placed in the forms. Exceptions to this can be made when placing concrete in cold-weather conditions. (NCDOT 2006)

2.8.7.3.1 Protection from Cold-Weather Conditions

Concrete should not be placed when the temperature in the shade is less than 35 ^{o}F (2 ^{o}C) without permission. If permission is given, the water and/or aggregate should be heated to a temperature at or below 150 ^{o}F (66 ^{o}C). The concrete can then be placed when its temperature is less than or equal to 80 ^{o}F (27 ^{o}C) and greater than or equal 55 ^{o}F (13 ^{o}C). (NCDOT 2006)

Aggregate that is frozen or has ice on it should not be used in mixing concrete. Also, concrete should not be placed on a foundation that is frozen. (NCDOT 2006)

Concrete should be protected by insulation or by heated enclosures when the air temperature in the shade at the time of placement is lower than 35 ^{o}F (2 ^{o}C), or when the air temperature in the shade is lower than 35 ^{o}F (2 ^{o}C) and the fresh concrete is less than 72 hours old. (NCDOT 2006)

2.8.7.4 Joints

2.8.7.4.1 Construction Joints

Transverse construction joints shall be placed in culverts that are longer than 70 ft (21 m). These joints should be oriented parallel to the main reinforcement in the slabs. The maximum allowable spacing for transverse construction joints is also 70 ft (21 m). (NCDOT 2007)

It is permissible for the bottom slab of a multiple barrel culvert to have longitudinal construction joints located 1 ft (305 mm) from the interior wall(s). The slab reinforcement can be spliced at these construction joints if the contractor desires. (NCDOT 2007)

2.8.7.4.2 Expansion Joints

All CIP culverts should have a 1 in. (25 mm) expansion joint in their wings. Filter fabric should be placed on the soil side of the joint to keep material from coming through. (NCDOT 2007) An example of an elevation view of a wingwall and the location of the expansion joint is shown in Figure 2-39, which is from Figure 9-6 of the *Structure Design Unit Design Manual* (2007).

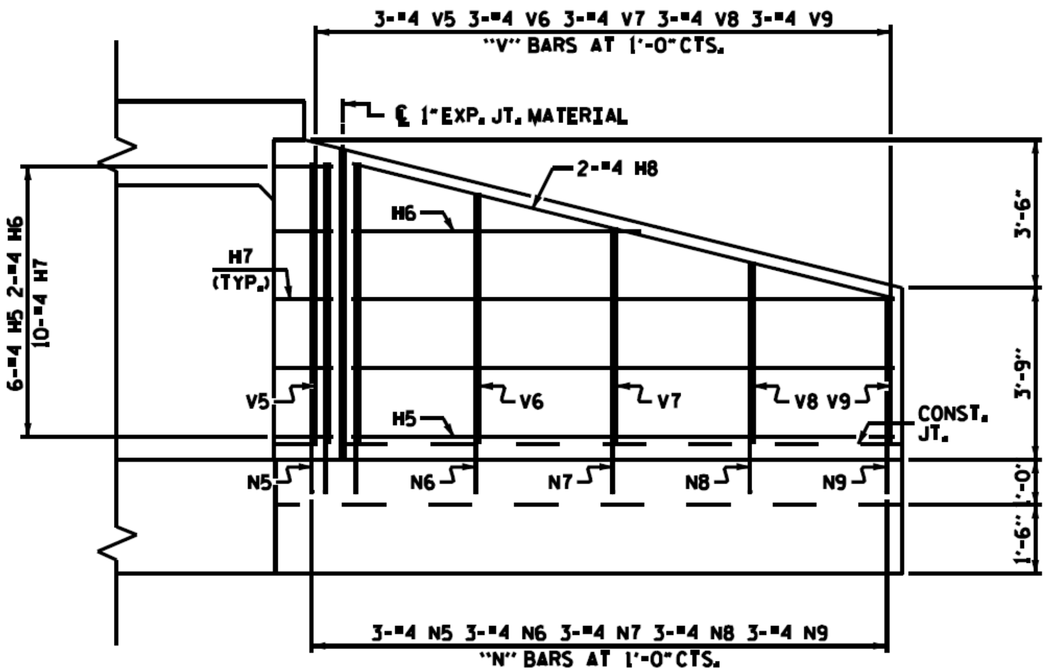


Figure 2-39: NCDOT Wingwall Expansion Joint Location (NCDOT 2007)

2.8.7.4.3 Joint Filler

Nonbituminous or bituminous joint filler can be used. The nonbituminous joint filler should meet AASHTO M 153 Type I, II, or III, and the bituminous joint filler should meet AASHTO M 213. (NCDOT 2006)

2.8.7.4.4 Joint Sealant

Hot-applied joint sealer and low modulus silicone sealant are the types of joint sealers that can be used. The hot-applied sealer should meet ASTM D 6690 specifications. The silicone sealant should meet ASTM D 5893 specifications. All expansion joints should be sealed with low modulus silicone sealant. Backer rods made of polyethylene foam or polyolefin foam should be used with the silicone sealant. (NCDOT 2006)

2.8.8 South Carolina Department of Transportation (SCDOT)

2.8.8.1 General Construction

Both precast and CIP reinforced concrete box culverts are used by the SCDOT (SCDOT 2007).

2.8.8.2 Concrete Protection from Environmental Conditions

2.8.8.2.1 Protection from Hot-Weather Conditions

A *Hot Weather Batching and Mixing Plan* should be developed to ensure that the temperature of the concrete is not greater than 90 ^{o}F (32 ^{o}C) when it is placed. This could include sprinkling aggregate with cool water, using Type II cement, and using cold water or shaved ice for mixing. The plan should meet the requirements set in ACI 305R. (SCDOT 2007)

A *Placing and Curing Plan* should also be implemented to achieve the same goal. It may include (SCDOT 2007):

- Spraying the forms and reinforcement with cool water,
- Scheduling concrete placement so that delays will be kept to a minimum,
- Pre-wetting the forms or subgrade to keep them from absorbing water from the concrete,

- Building windbreakers to keep the exposed concrete surfaces from drying,
- Screeding, floating, and starting the concrete curing process as soon as the concrete is placed, and
- Providing evaporative cooling by using water-curing techniques.

2.8.8.2.2 Protection from Cold-Weather Conditions

When the temperature is below 35 ^{o}F (2 ^{o}C), a *Cold Weather Batching and Mixing Plan* should be developed. At a minimum it should include:

- Uniformly heating the aggregate using steam or dry heat,
- The mixing water can also be heated but it should not be above 170 ^{o}F (77 ^{o}C) when it is poured into the mixer,
- Ensuring that the concrete temperature is at least 50 ^{o}F (10 ^{o}C) when it is placed, and
- Avoiding using aggregates with frozen particles.

A *Placing and Curing Plan* should also be implemented to make sure that the air temperature around the concrete is not lower than 50 ^{o}F (10 ^{o}C). (SCDOT 2007)

2.8.8.3 Joints

2.8.8.3.1 Construction Joints

Longitudinal construction joints between the base of the culvert and the walls should be keyed as shown in Figure 2-40. Joints in interior walls or midwall joints should also be similar to Figure 2-40 if possible. (SCDOT 2009)

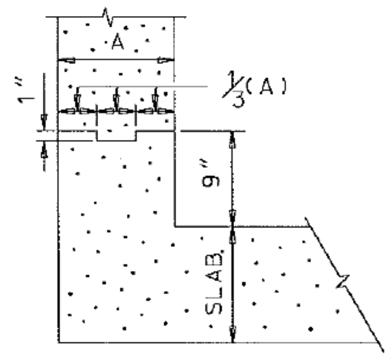


Figure 2-40: SCDOT Longitudinal Construction Joint Detail for Wall to Base Connections (SCDOT 2009)

2.8.8.3.2 Joint Waterproofing

Waterstops made of flexible polyvinyl chloride should be used in expansion joints (SCDOT 2007).

2.8.8.3.3 Joint Filler

Joint filler for expansion joints can be preformed, hot-poured elastic, or a cold-applied sealant. The preformed filler should meet AASHTO M 213 or ASTM D 6690 Type I requirements. The hot-poured filler should meet ASTM D 6690 Type I requirements. The cold-applied filler should meet ASTM C 920 requirements. A polyethylene backer rod should be used with the cold-applied sealant. (SCDOT 2007)

2.8.9 Tennessee Department of Transportation (TDOT)

2.8.9.1 General Construction

The TDOT uses both precast and CIP reinforced concrete box culverts (TDOT 2006).

A typical cross section of a double barrel CIP box culvert used by the TDOT is shown in Figure 2-41.

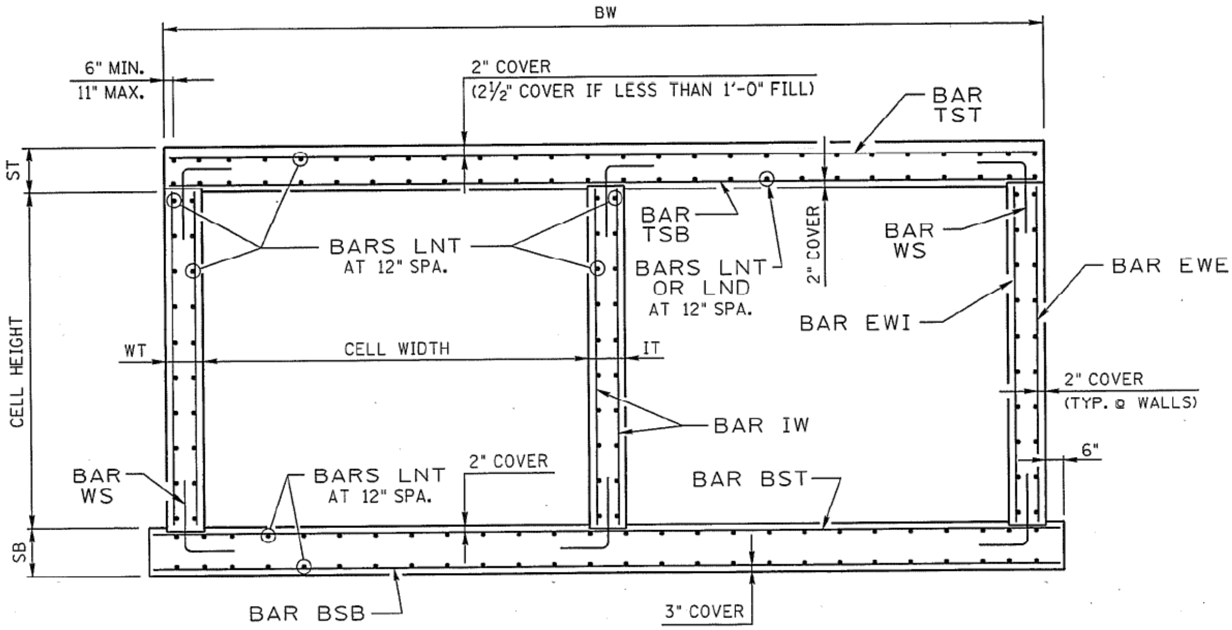


Figure 2-41: Typical TDOT Cast-in-Place Box Bridge Cross Section (TDOT 2000c)

2.8.9.2 Wall Height Construction Limitations

For CIP culverts 6 ft (2 m) high or less, the top slab and side walls can be built as a monolith. When this is done, construction joints should be normal to the culvert's axis and upright. For culverts that are greater than 6 ft (2 m) high, the side walls should be placed first and then let set for a minimum of 4 hours before placing the top slab. When this is done, there should be keys in the side walls for the purpose of anchoring the top slab. (TDOT 2006)

2.8.9.3 Concrete Protection from Environmental Conditions

2.8.9.3.1 Protection from Hot-Weather Conditions

When conditions consist of high temperature, low humidity, and/or high winds, the fresh concrete should be protected to prevent drying shrinkage cracking from occurring. As a rule, no concrete will be placed when the evaporation rate is greater than 0.2 lb/ft^2/hr (1 kg/m^2/hr). (TDOT 2006)

2.8.9.3.2 Protection from Cold-Weather Conditions

If the ambient temperature is below 35 ^{o}F (2 ^{o}C) after the concrete has been placed, the air temperature around the concrete should be kept at or above 45 ^{o}F (7 ^{o}C). Also, the concrete temperature should not be greater than 80 ^{o}F (27 ^{o}C). These conditions should be kept until 120 hours after the concrete is placed. (TDOT 2006)

2.8.9.4 Joints

2.8.9.4.1 Construction Joints

For stage construction joints where the fill is not greater than 3.5 ft (1.1 m) high, TDOT drawing STD-15-2 (2000a) states, "When a box or slab bridge must be stage constructed such that the construction joint is not perpendicular to the bridge, the stage construction joint shall be a plain butt joint, and no reinforcement shall extend across the joint." Dowel bars should also be used as shown in Figure 2-42. These joints should not be located underneath a traffic lane. (TDOT 2000a)

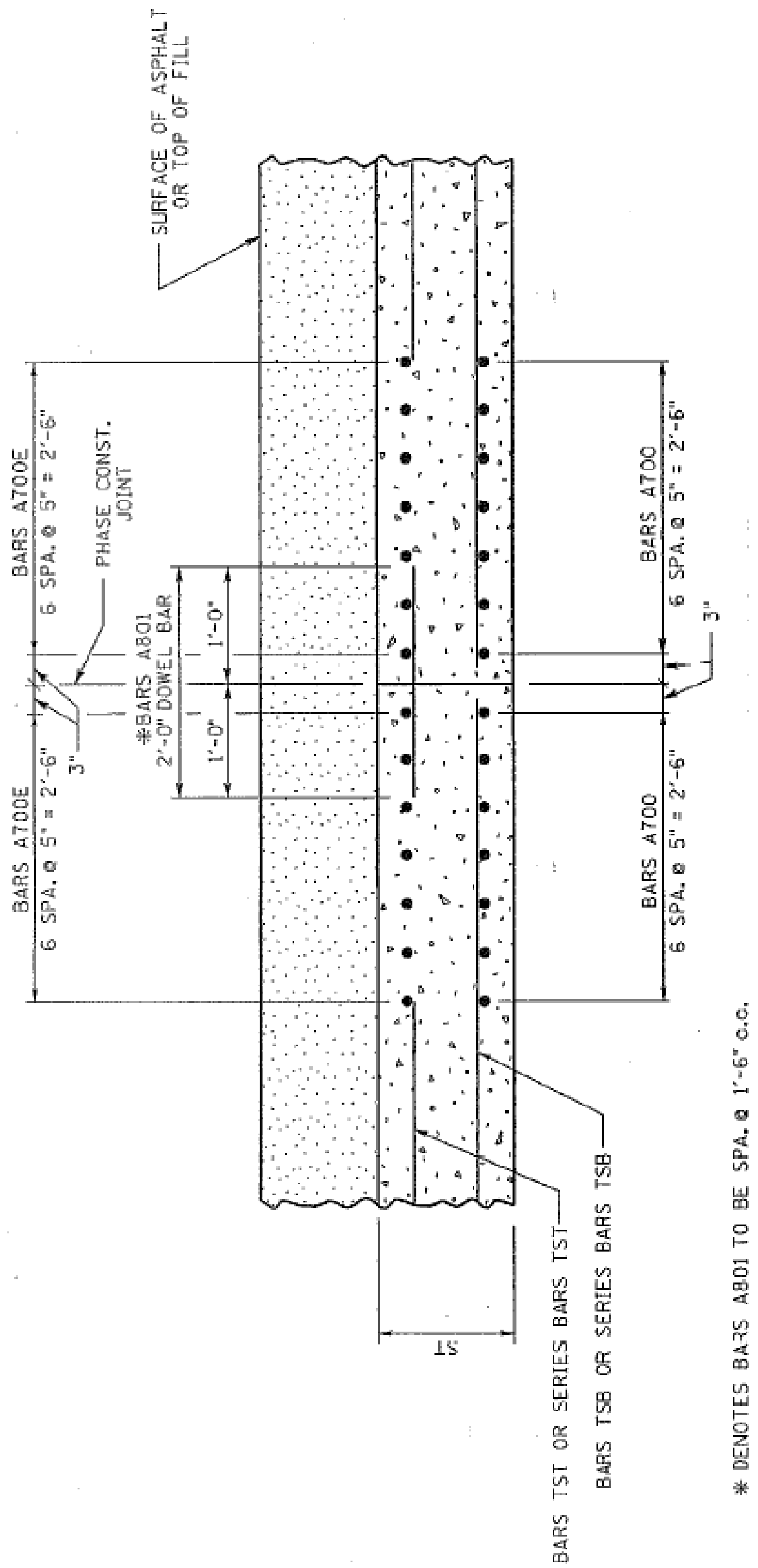


Figure 2-42: TDOT Box Culvert Stage Construction Joint (TDOT 2000b)

The construction joints between the top and bottom slabs and the exterior walls are shown in Figure 2-43. The construction joints between the top and bottom slabs and the interior walls are shown in Figure 2-44. The construction joints between the top and bottom slabs and the interior walls that should be used when the fill height is greater than 10 ft (3.0 m) are shown in Figure 2-45. The steel is continuous through the joint in each of these cases (TDOT 2000c).

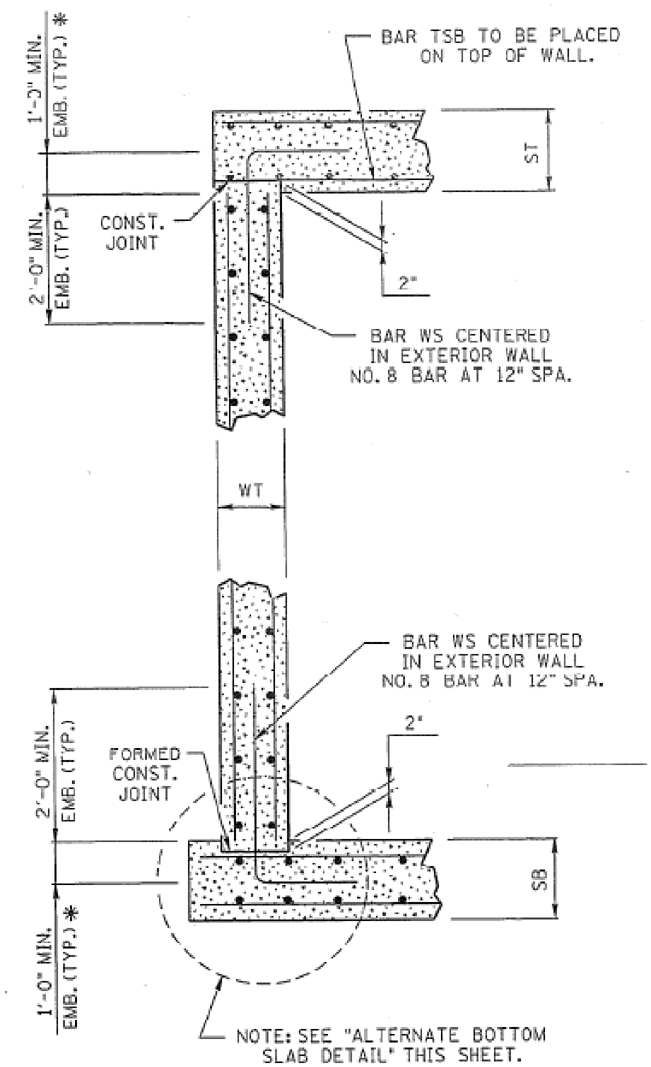


Figure 2-43: TDOT Construction Joints between Slabs and Exterior Walls (TDOT 2000c)

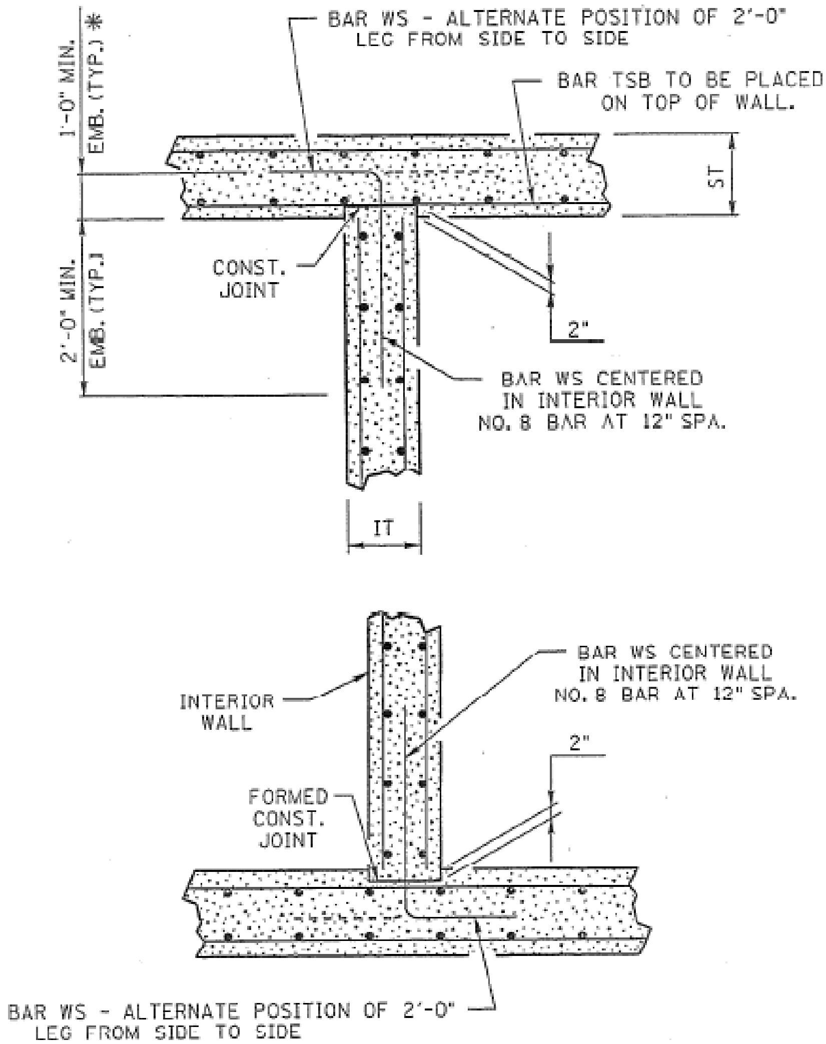


Figure 2-44: TDOT Construction Joints between Slabs and Interior Walls (TDOT 2000c)

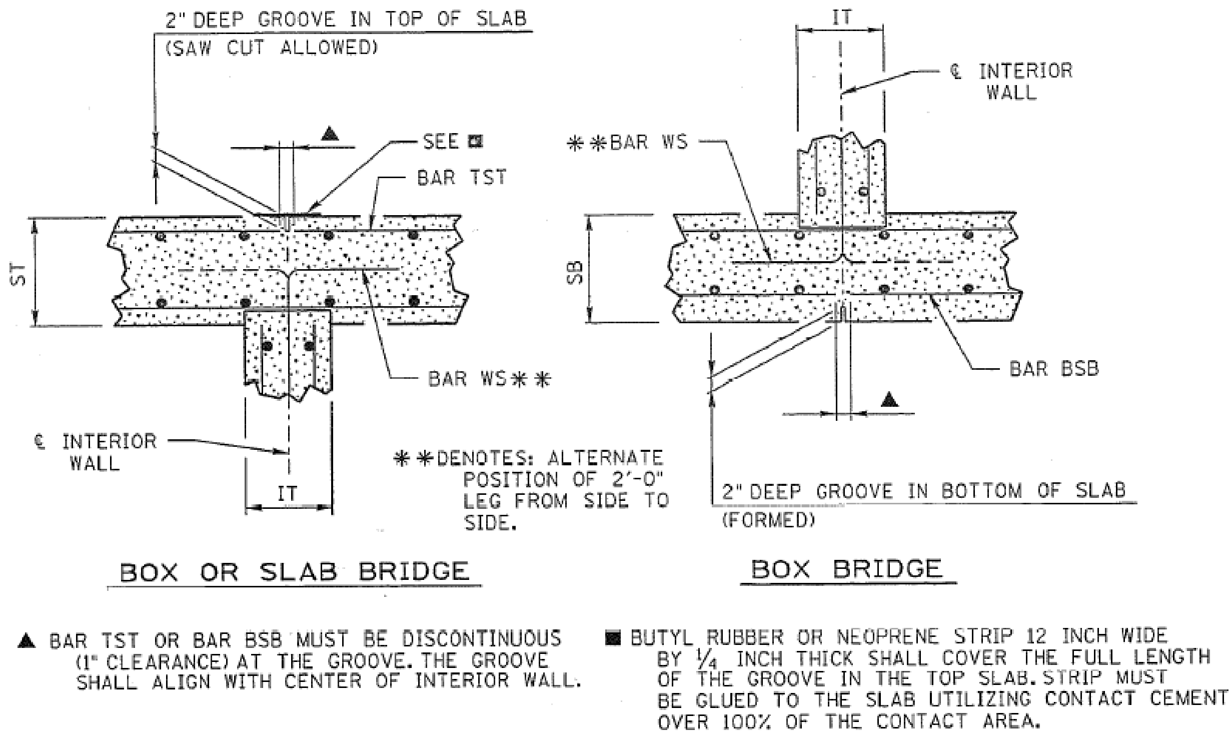


Figure 2-45: TDOT Interior Construction Joints between Walls and Slabs when the Fill is Greater than 10 Feet (3.0 m) (TDOT 2000c)

2.8.9.4.2 Contraction Joints

Transverse contraction joints shall be used as transverse stage construction joints in box culverts when the fill height is more than 3.5 ft (1.1 m) (TDOT 2000a). The contraction joints used in box culverts should be plain butt joints. *Reinforcement should not be continuous through these joints*. These joints should be spaced at 30 to 40 ft (9.1 to 12 m), and should be placed at points where the box section changes, when possible. The contraction joints should run parallel to the main reinforcement in the slab, but not necessarily normal to the axis of the culvert. (TDOT 2006) When the top slab of the box culvert is going to be used as riding surface for traffic, no contraction joint should be used. (TDOT 2000a).

Vertical contraction joints are also located in the wingwall of the culvert. They are located at mid-length of the wingwall when it is longer than 30 ft (9.1 m). (TDOT 2000d) The location of the contraction joint on the wingwall is shown in Figure 2-46, and a detail of a wingwall contraction joint is shown in Figure 2-47.

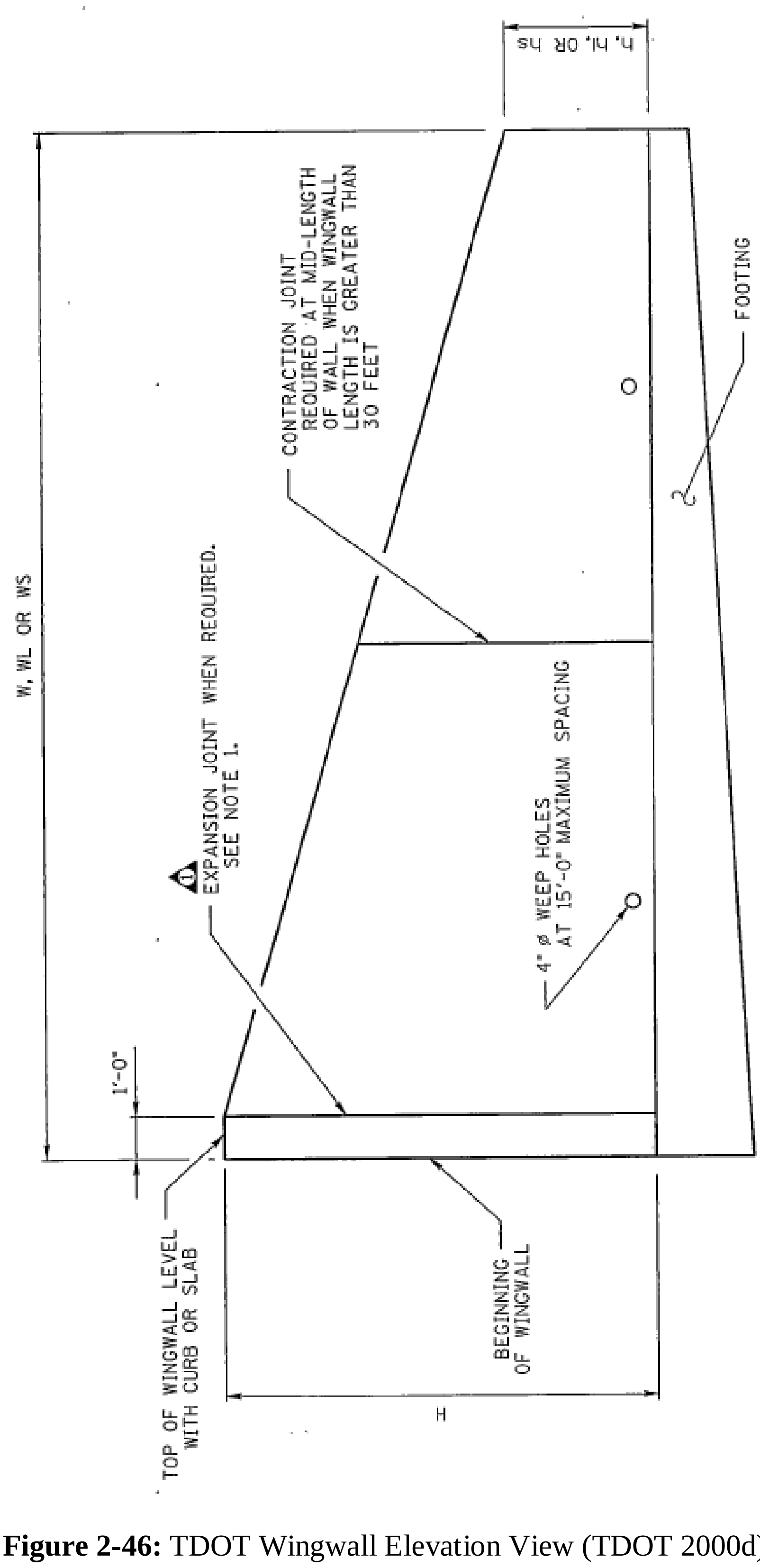


Figure 2-46: TDOT Wingwall Elevation View (TDOT 2000d)

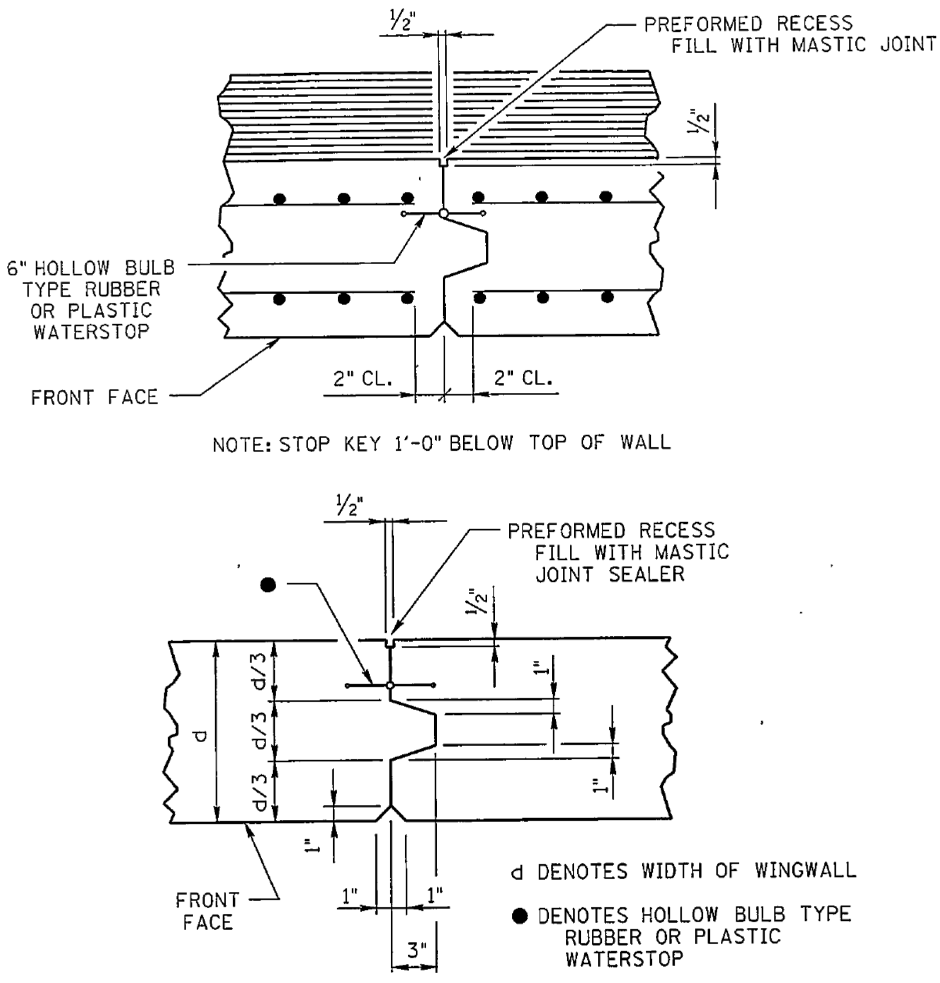


Figure 2-47: TDOT Wingwall Vertical Contraction Joint (TDOT 2000d)

2.8.9.4.3 Expansion Joints

Note 1 in Figure 2-46 states that vertical expansion joints are required when the wingwall is longer than 15 ft (4.6 m). *The joint should be only in the wing wall and not in the footing*. They are located at the junction between the outer culvert wall and the wingwall. *Reinforcing bars shall not be continuous through the expansion joint*. One of the ends of the dowel bars used in the joint should be covered with tarpaper and also have a tar-paper end cap. (TDOT 2000d) The location of the expansion joint on a wingwall is shown in Figure 2-46, and a detail for a wingwall expansion joint is shown in Figure 2-48.

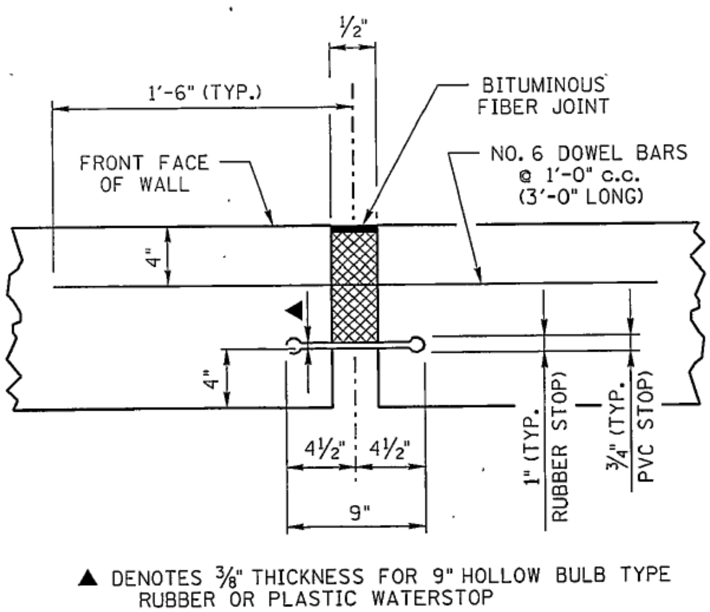


Figure 2-48: TDOT Vertical Wingwall Expansion Joint (TDOD 2000d)

2.8.9.4.4 Joint Waterproofing

Waterstops can be made from copper, natural rubber, synthetic rubber, or polyvinyl chloride (PVC). They should be uniform throughout, dense, and without imperfections. (TDOT 2006) See Figures 2-46 and 2-47 for waterstop illustrations.

2.8.9.4.5 Joint Filler

Preformed nonbituminous or bituminous joint filler can be used. The nonbituminous joint filler should meet AASHTO M 153 Type I, II, or III specifications, and the bituminous joint filler should meet AASHTO M 213 specifications. (TDOT 2006)

2.8.9.4.6 Joint Sealant

Longitudinal and transverse joints may be sealed using a silicone sealant or a hot-poured elastic type sealant that meets ASTM D 3405 specifications. The hot-poured sealant must be made of virgin synthetic and/or reclaimed rubber combined with tacifiers, plasticizers, and asphalt. A backer rod may also be used along with the sealant. The sealant and backer rod shall not be allowed to bond together. (TDOT 2006)

2.8.10 Virginia Department of Transportation (VDOT)

2.8.10.1 General Construction

The VDOT allows contractors to use CIP and precast reinforced concrete box culverts (VDOT 2007). A typical single barrel CIP box culvert used by the VDOT is shown in Figure 2-49.

Each wingwall should be built as a monolith if possible. If it is not possible, the construction joints should be horizontal. (VDOT 2007)

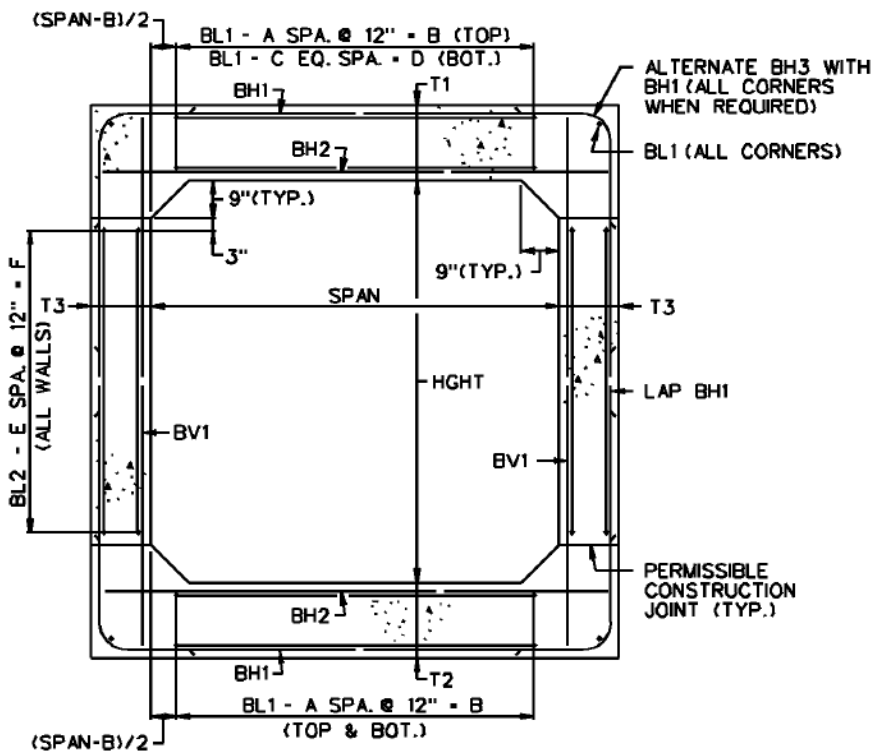


Figure 2-49: Typical VDOT Single Barrel Cast-in-Place Box Culvert Cross Section (VDOT 2008)

2.8.10.2 Joints

2.8.10.2.1 Construction Joints

Box culverts more than 35 ft (11 m) long shall have construction joints spaced at no more than 25 ft (7.6 m) and located no more than 30 ft (9.1 m) from the ends (VDOT 2008).

If consecutive courses need to be bonded, keys should be formed. They should be formed by inserting beveled wood strips that have been saturated with water. Dowel bars may be used instead of keys if the engineer chooses. (VDOT 2007)

2.8.10.2.2 Joint Waterproofing

Waterstops can be made of copper, neoprene, or polyvinyl chloride (PVC) (VDOT 2007). A 6 x 3/8in. (152 x 76 mm) Dumbbell PVC waterstop should be used in the joints between the culvert and the wingwall (VDOT 2008).

2.8.10.2.3 Joint Filler

Joint fillers can be preformed, expanded rubber, PVC, PE, or sponge rubber. The preformed filler should meet AASHTO M213 requirements, the expanded rubber filler should meet ASTM D 1056 requirements, the preformed neoprene filler should meet ASTM D 1056 Grade 2BS requirements, the PVC and PE filler should meet ASTM D 1667, and the sponge-rubber filler should meet AASHTO M 153 requirements. (VDOT 2007)

2.8.10.2.4 Joint Sealant

Joint sealants can be hot-poured, silicone, or preformed elastomeric joint sealants. The hot-poured sealant can be an asphalt sealer, meeting ASTM D 6690 Type II specifications, or an elastomeric joint sealer meeting ASTM D3406 specifications. The elastomeric joint sealer should only be used for longitudinal joints. Bond breakers should be used with the silicone sealant. The bond breaker can be a backer rod (made of closed-cell expanded polyethylene foam or closed cell expanded polyolefin foam) or bond-breaking tape (made from extruded polyethylene). (VDOT 2007)

2.8.11 West Virginia Department of Transportation (WVDOT)

2.8.11.1 General Construction

The WVDOT uses box culverts that can be designed as CIP or precast (WVDOT 2004).

Culverts with a clear span of 16 ft (4.9 m) or less are typically used. Single barrel culverts should be used whenever it is possible, and triple barrel culverts should generally be avoided due to high construction and maintenance costs associated with them. (WVDOT 2004)

2.8.11.2 Concrete Protection from Environmental Conditions

The concrete temperature should be no lower than 50 ^{o}F (10 ^{o}C) and no greater than 85 ^{o}F (29 ^{o}C) when it is placed (WVDOT 2000).

2.8.11.2.1 Protection from Hot-Weather Conditions

When the air temperature in the shade reaches 85 ^{o}F (29 ^{o}C) the concrete temperature should be monitored, and when the concrete temperature reaches 85 ^{o}F (29 ^{o}C) the concrete should be poured within an hour of the mixing water being introduced. If the concrete temperature rises to 90 ^{o}F (32 ^{o}C) the mixing water and/or aggregate should be cooled to lower the concrete temperature. Crushed or flaked ice may be used also. Concrete should never be placed when the temperature at the end of mixing exceeds 90 ^{o}F (32 ^{o}C). (WVDOT 2000)

It is also important to make sure that the surface of the concrete is kept wet during the curing process to prevent shrinkage cracking (WVDOT 2002).

2.8.11.2.2 Protection from Cold-Weather Conditions

Cold weather provisions go into effect when the plastic concrete has a temperature less than 55 ^{o}F (13 ^{o}C). To keep the concrete at acceptable temperatures, the aggregate and/or the mixing water may be heated. The aggregate or water should not have a temperature higher than 150 ^{o}F (66 ^{o}C) when it is poured into the mixture. Any materials that are frozen should not be used in the concrete mixture. (WVDOT 2000)

When the concrete is curing, the surface of the concrete must be kept at a temperature above 35 ^{o}F (2 ^{o}C). However, days when the concrete surface temperature is below 50 ^{o}F (10 ^{o}C) cannot be counted as curing days. (WVDOT 2002) Insulated forms may be used to protect the concrete during this period (WVDOT 2000). Cold weather protection should be taken off in a way to prevent concrete surface temperature from dropping more than 20 ^{o}F (-7 ^{o}C) in a 24-hour period (WVDOT 2002).

2.8.11.3 Joints

2.8.11.3.1 Joint Waterproofing

Waterstops can be made of polyvinyl chloride (PVC) or rubber, and they should be free from imperfections, dense, and the same throughout (WVDOT 2000).

2.8.11.3.2 Joint Filler

Joint fillers for the expansion joint can be preformed. They should be non-extruding and either resilient no bituminous or resilient bituminous types. The non-bituminous type should meet AASHTO M 513 requirements and the bituminous type should meet AASHTO M 213 requirements. (WVDOT 2000)

2.8.11.3.3 Joint Sealant

Joint sealants can be hot-poured, silicone, or a mortar. The hot-poured sealant should meet ASTM D 3405 requirements, and the silicone sealant should have a back-up material that prevents the sealant from moving to the bottom of the joint. (WVDOT 2000)

Chapter 3

Box Culvert Crack Condition Survey

3.1 Introduction

As stated in Section 1.2, one of the objectives of this project was to examine if transverse cracking similar to that found in the Anniston Eastern Bypass (AEB) project was experienced in other cast-in-place (CIP) reinforced concrete box culverts in Alabama. This was achieved by doing culvert crack condition surveys of several CIP reinforced concrete box culverts throughout Alabama. The surveys consisted of a team walking through culverts and documenting the location and width of every visually observable transverse crack. Culverts were visited in five ALDOT divisions. Construction documents that contained the time of placement and environmental conditions during the placement of each section were also gathered from ALDOT for each of the surveyed culverts if they were available.

3.2 Survey Procedure

In the culvert crack condition surveys, crack comparators were used to measure crack widths, a measuring wheel was used to measure the distance from the beginning of the culvert to the crack location, and construction crayons were used to trace the crack and to write the crack width next to the crack. A crack culvert survey documentation sheet was also prepared for each culvert survey visits, which is shown in Appendix A.

When the box culvert crack condition surveys were begun, the end of the culvert (north, south, east, or west) that was entered was determined first. Then the wall on which the measuring wheel would be used was decided. The measuring wheel was always zeroed at the beginning of the culvert. The width and station of every transverse crack and transverse joint in the box culvert were then documented. In addition, the location in the cross section of the culvert (walls, ceiling, or base) that the crack occurred was also documented. The crack widths were recorded in 10^{-3} in. for simplicity (e.g. a 0.012 in. wide crack was reported as 12 x 10^{-3} in.). The cracks widths were measured at the location of the crack that was the widest. Openings in all joints were recorded. If a joint experienced no movement or opening, it was designated as closed or tight. Also, if a joint or crack was patched, sealed, or covered in some way it was designated as "CM", or could not measure. Longitudinal cracks and plastic shrinkage cracks were also documented, as well as any corrosion stains, efflorescence, exposed reinforcement, spalls, popouts, scaling, or previous repairs to the concrete.

3.3 Culverts Visited

Minor distress, such as corrosion and efflorescence, were seen in almost all of the culverts surveyed, as well as some minor longitudinal and plastic shrinkage cracks. However, only major distress signs, transverse cracks, or conditions unique to each culvert will be mentioned in this chapter. For full crack condition survey results see Appendix A. The locations of the culverts visited, as well as the ALDOT division locations are shown in Figure 3-1. The geometry, length, and fill height for all of the culverts surveyed can be seen in Table 3-1.

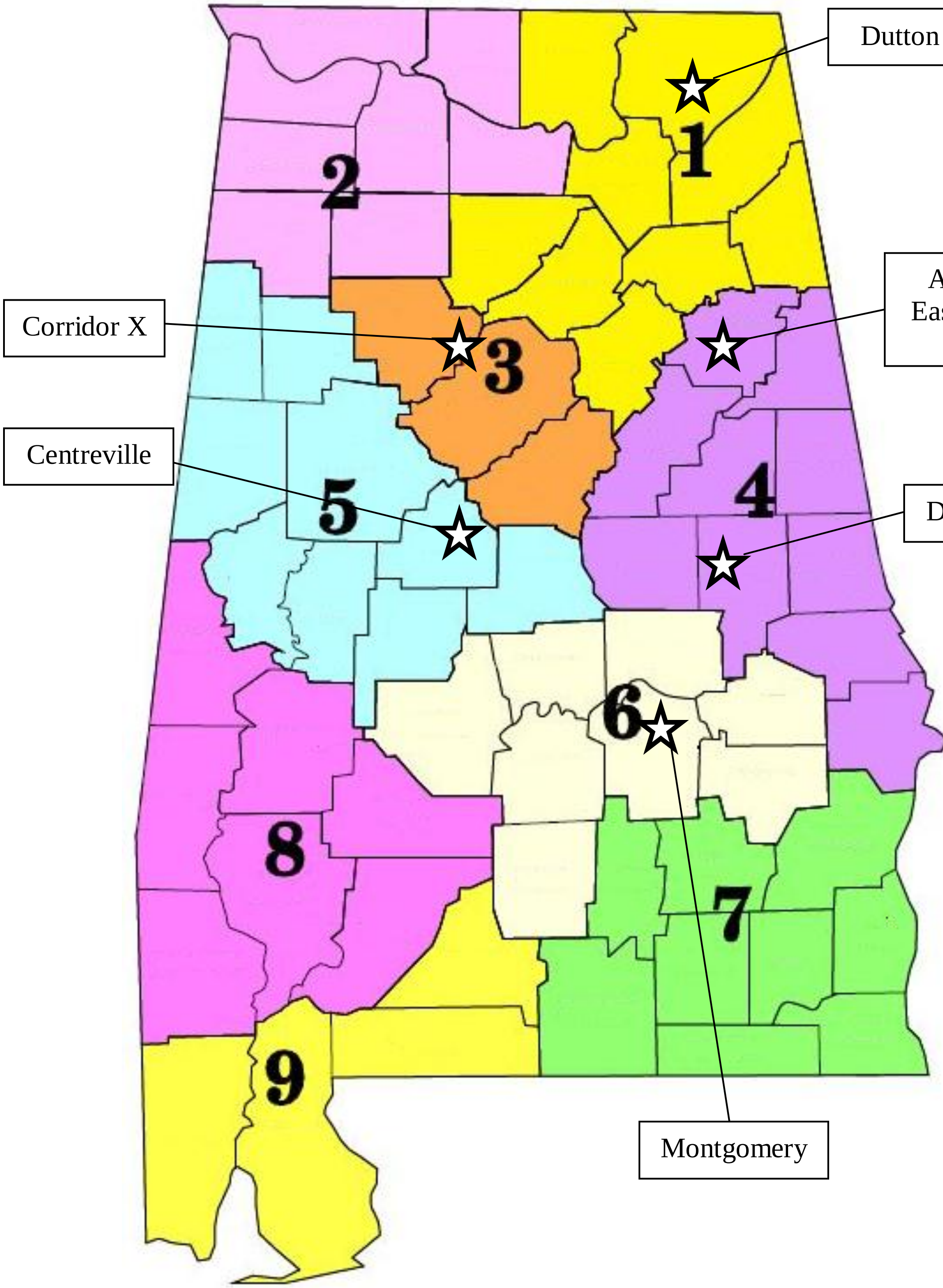


Figure 3-1: Culvert Survey Locations (ALDOT 2011a)

Table 3-1: Surveyed Culvert Information

Surveyed Culvert Information						
Culvert ID	Size (No. Barrels x Width x Height)	Length	Maximum Fill Height	Ceiling Thickness	Interior Wall Thickness	Exterior Wall Thickness
AEB 149+60 (C)	1x8'x8'	1,005'	56’	17”	--	14”
AEB 162+90 (D)	1x6'x6'	355'	36’	13.5”	--	11.5”
AEB 175+70 (E)	1x8'x6'	508'	52’	16.5”	--	10.5”
AEB 240+37 (J)	2x8'x8'	892'	59'	18.5"	9"	15"
AEB 257+69 (I)	1x6'x6'	625'	78'	18"	--	13"
Centreville 1808+98	3x12'x7'	286'	12'	14"	6"	10.5"
Corridor X 4877+13	3x8'x10'	901'	124'	2'-10"	12"	2'-11"
Corridor X 4959+43	3x8'x10'	945'	110'	2'-8"	12"	2'-10"
Corridor X Exit 85	3x6’x6’	> 900’	N/A	N/A	N/A	N/A
Dadeville 45+31.55	3x10'x10'	305'	32'	16.5"	7"	12"
Dutton 548+23	1x8'x8'	929'	114’-120’	24”	N/A	19.5”
I-85 North	2x10’x7’	122’	N/A	N/A	N/A	N/A
I-85 South	2x12’x7’	287’	N/A	N/A	N/A	N/A

3.3.1 Anniston East Bypass (AEB)

The culverts surveyed were located in Anniston in ALDOT’s Fourth Division. The culvert sites were under the section of the Anniston East Bypass that was 700 ft (213 m) north of Choccolocco Road and 1,500 ft (457 m) south of Lake Yahou.

3.3.1.1 AEB Culvert at 149+60 (Culvert C)

The condition survey of AEB Culvert at 149+60, also known as Culvert C, was performed on July 12, 2010**.** This culvert was built with transverse construction joints that contained continuous longitudinal reinforcement and not contraction joints. The average joint spacing used was 48 ft (15 m). The vee joint in Figure 1-3 was specified in the plans. The measuring wheel was zeroed on the north wall and the end of entry was the west end. The cross section of AEB Culvert at 149+60 can be seen in Figure 3-2, and the entrance to the culvert is shown in Figure 3-3.

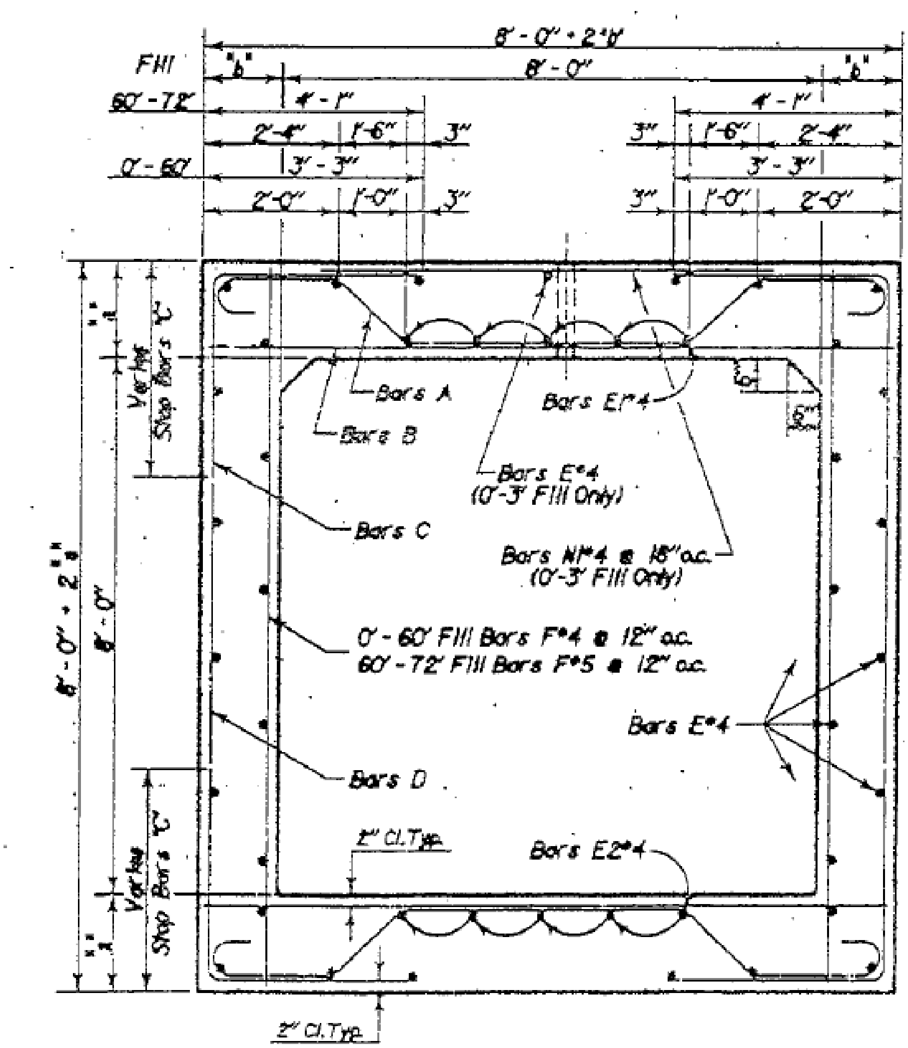


Figure 3-2: AEB Culvert at 149+60 Cross Section (ALDOT 1986)

Figure 3-3: AEB Culvert at 149+60 Entrance

The widest crack observed was 1/8 in. (3.2 mm) in the north wall at stations 308 ft 0 in. and 314 ft 0 in., and the widest construction joint opening was 1/4 in. (6.4 mm) at station 293 ft 3 in. in the base. See Figure 3-4 for the wall crack at station 308 ft 0 in. Several of the construction joints were patched with mortar. At station 293 ft 3 in. the base of the construction joint had an opening of 1/4 in. (6.4 mm) and water was running through this crack into the supporting base. The survey was stopped at the top of the slope at station 1,003 ft 0 in. because the rest of the culvert was under water. Longitudinal chamfer cracks were observed on both sides of the culvert barrel.

Figure 3-4: 1/4 in. (6.4 mm) Wide Transverse Crack in AEB Culvert at 149+60 at Station 308 ft 0 in. in the North Wall

3.3.1.2 AEB Culvert at 162+90 (Culvert D)

The condition survey of AEB Culvert at 162+90, also known as Culvert D, was performed on July 12, 2010. This culvert was built with transverse construction joints that contained continuous longitudinal reinforcement and not contraction joints. The average joint spacing used was 53 ft (16 m). The measuring wheel was started on the north wall and the end of entry was the east end. The inside of the barrel of this culvert can be seen in Figure 3-5. The plans for this culvert could not be obtained.

The widest crack observed was 1/8 in. (3.2 mm) at multiple locations in the base and both walls. See Figure 3-6 for one of these cracks found at station 179 ft 0 in. in the base. The widest construction joint opening was at station 153 ft 6 in., which had an opening greater than 0.08 in. (2 mm). Longitudinal chamfer cracks were observed on both sides of the culvert barrel.

Figure 3-5: Inside of AEB Culvert at 162+90

Figure 3-6: 1/4 in. (3.2 mm) Wide Transverse Base Crack at Station 179 ft 0 in. in AEB Culvert at 162+90

3.3.1.3 AEB Culvert at 175+70 (Culvert E)

The condition survey of AEB Culvert at 175+70, also known as Culvert E, was performed on July 12, 2010. This culvert was built with transverse construction joints that contained continuous longitudinal reinforcement and not contraction joints. The average joint spacing used was 52 ft (16 m). The vee joint in Figure 1-3 was specified in the plans. The measuring wheel was started on the north wall and the end of entry was the east end. The cross section of AEB Culvert at 175+70 can be seen in Figure 3-7, and the entrance is shown in Figure 3-8.

The widest crack observed was 3/16 in. (4.8 mm) wide at station 148 ft 7 in. in both walls and the ceiling. One of the wider transverse cracks can be seen in Figure 3-9. A vertical reinforcement bar was visible through a wide transverse crack on the north wall at station 148 ft 7 in. The widest construction joint opening was 0.08 in. (2 mm) wide in the base and north wall at station 112 ft 3 in. Longitudinal chamfer cracks were observed in this culvert. The chamfer cracks ranged from hairline to 0.016 in. (0.41 mm) wide.

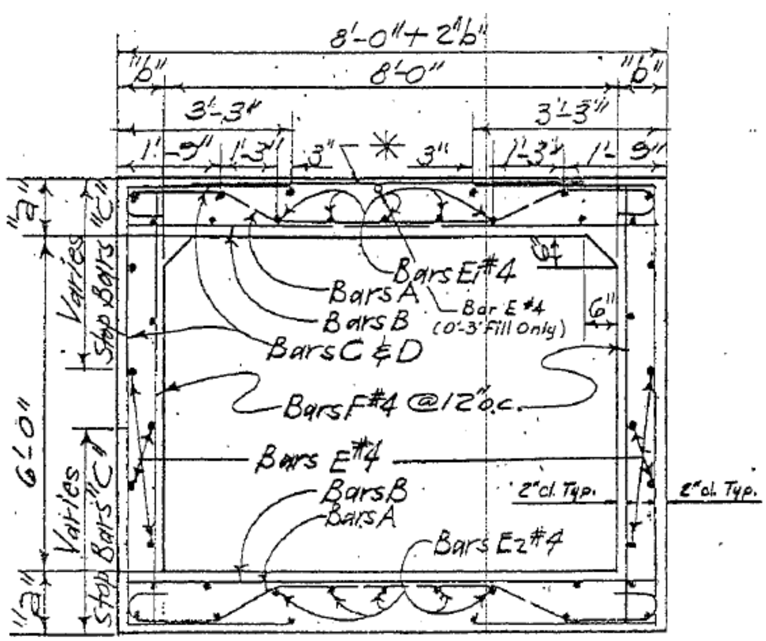


Figure 3-7: AEB Culvert at 175+70 Cross-Section (ALDOT n.d.b)

Figure 3-8: AEB Culvert at 175+70 Entrance

Figure 3-9: Transverse Wall and Base Crack in AEB Culvert at 175+70 that is greater than 0.08 in. (2 mm) wide

3.3.1.4 AEB Culvert at 240+37 (Culvert J)

The condition survey of AEB Culvert at 240+37, also known as Culvert J, was performed on November 9, 2010. This culvert was built with transverse construction joints that contained continuous longitudinal reinforcement and not contraction joints. The average joint spacing used was 50 ft (15 m). The vee joint in Figure 1-3 was specified in the plans. The measuring wheel was started on the north wall, the end of entry was the east end, and the north barrel was surveyed. The north wall was an exterior wall and the south wall was an interior wall. The cross section of AEB Culvert at 240+37 is shown in Figure 3-10.

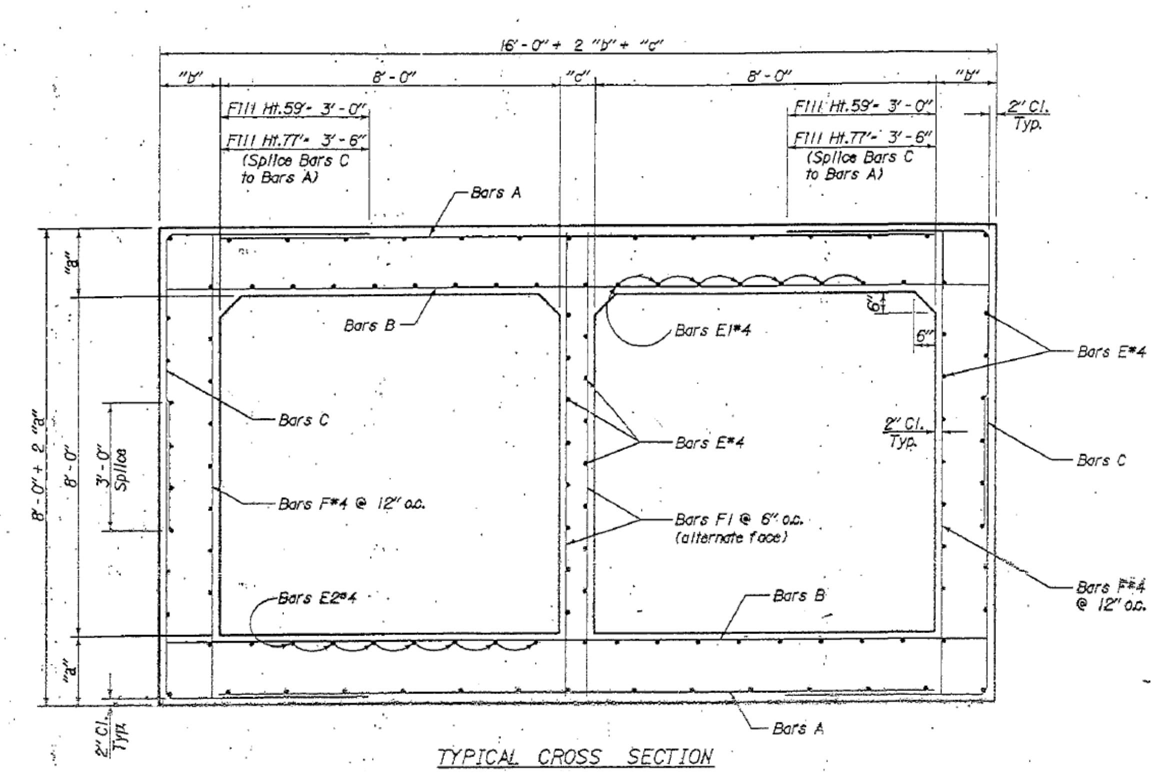


Figure 3-10: AEB Culvert at 240+37 Cross-Section (ALDOT n.d.a)

Many of the cracks in Culvert J had been repaired when the research team visited the culvert site, and the crack widths could not be measured; however some of these repaired cracks had already started to reopen. Because of this, a previous ALDOT crack survey was used for all of the data. The ALDOT survey is included in Appendix A.

The widest crack observed was 3/16 in. (4.8 mm) wide at station 595 ft 0 in. in the ceiling, both walls, and the base. The widest joint opening was also 3/16 in. (4.8 mm) at station 69 ft 0 in. in the celling, both walls, and the base. A picture of a base crack found in the culvert can be seen in Figure 3-11. Cracks were also observed in the wingwalll of the culvert, as shown in Figure 3-12. Longitudinal cracs

Figure 3-11: Base Crack from AEB Culvert at 240+37

Figure 3-12: Wingwall Cracks in AEB Culvert at 240+37

3.3.1.5 AEB Culvert at 257+69 (Culvert I)

The condition survey of AEB Culvert at 257+69, also known as Culvert I, was performed on July 12, 2010. The measuring wheel was started on the north wall and the end of entry was the east end. The cross section of AEB Culvert at 257+69 is shown in Figure 3-13, and the entrance can be seen in Figure 3-14.

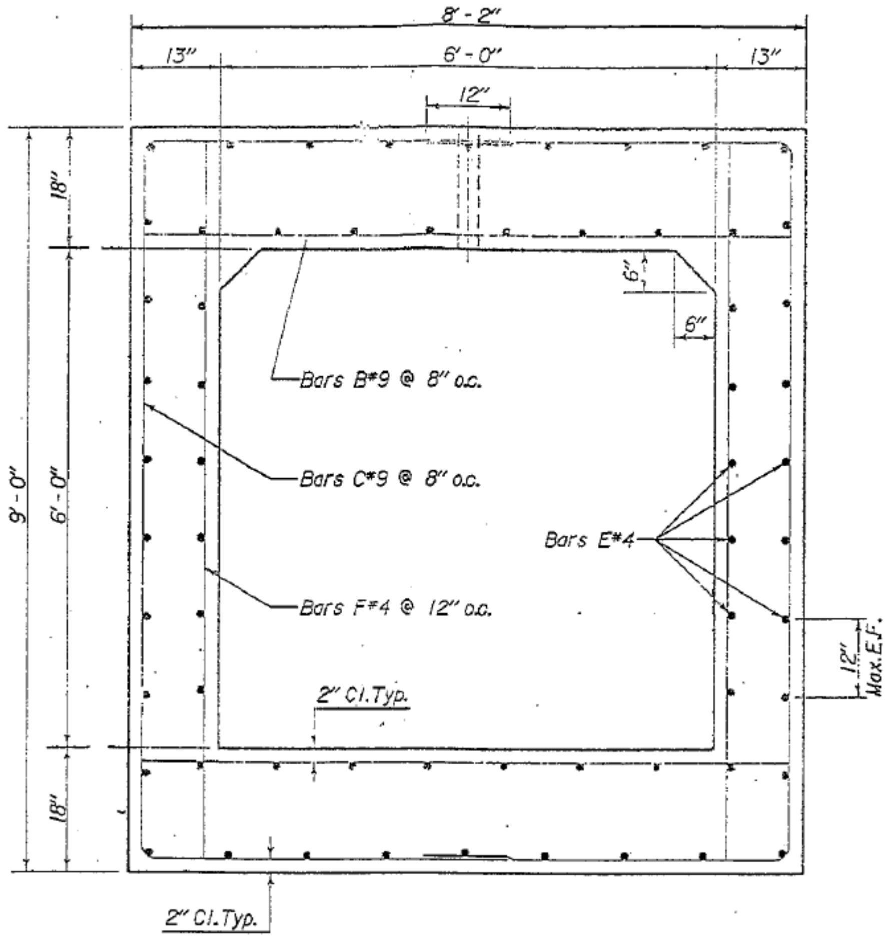


Figure 3-13: AEB Culvert at 257+69 Cross-Section (ALDOT 2001)

Figure 3-14: AEB Culvert at 257+69 Entrance

This culvert had contraction joints as detailed in Figure 3-15. The average joint spacing used was 49 ft (15 m). This was the only culvert in the AEB project detailed with a contraction joint and a shear key. Sealant was used in the base and walls at most of these contraction joints (see Figure 3-16a). However, the base was patched at stations 149 ft 3 in., 246 ft 10 in., and 440 ft 10 in., and wood was in the joint at the base at stations 99 ft 3 in. and 295 ft 7 in. (see Figure 3-16b). Stations 344 ft 8 in. and 392 ft 3 in. had construction joints instead of contraction joints.

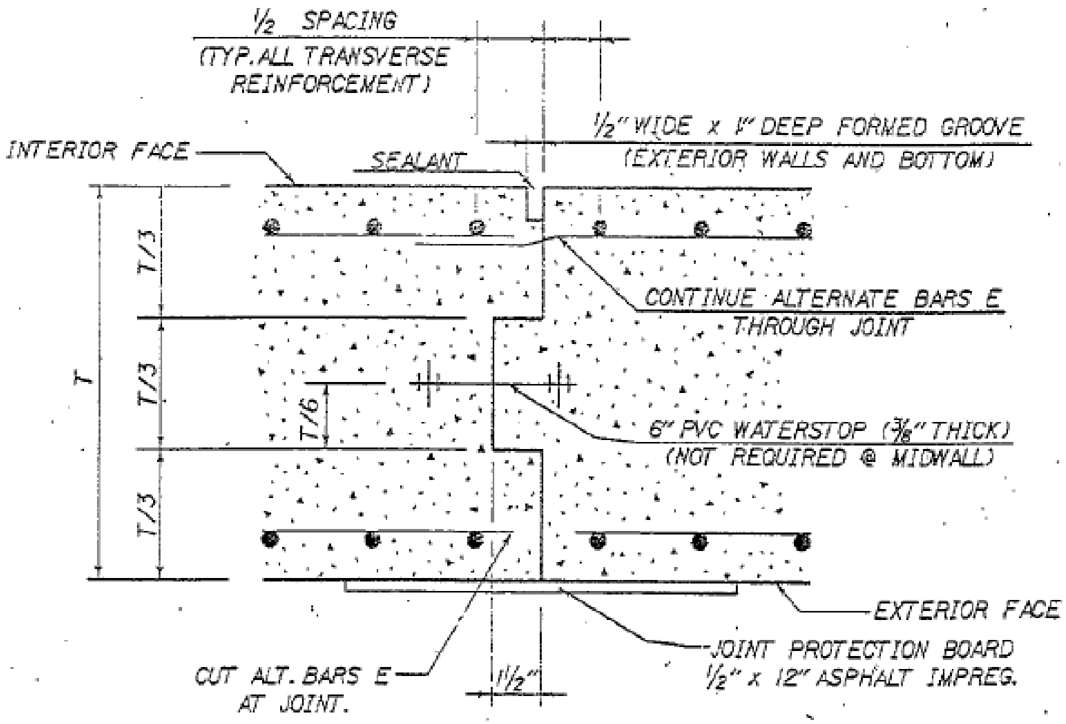


Figure 3-15: Transverse Contraction Joint Used in AEB Culvert at 257+69 (ALDOT 2001)

(a) (b)

Figure 3-16: (a) Contraction Joint and Sealant in AEB Culvert at 257+69;

(b) Wood Strip in a Contraction Joint in AEB Culvert at 257+69

The widest crack observed was 1/8 in. (3.2 mm) wide in the base at station 510 ft 9 in., and the widest construction joint opening was 3/16 in. (4.8 mm) at station 440 ft 10 in. in the ceiling.

3.3.2 Centreville

3.3.2.1 Centreville Culvert at 1808+98

This condition survey was performed on October 6, 2011. Centreville Culvert at 1808+98 was built with transverse construction joints that contained continuous longitudinal reinforcement and not contraction joints. The average joint spacing used was 51 ft (16 m). The vee joint in Figure 1-3 was specified in the plans. It is located east of County Road 20 on US Highway 82 in Centreville. Each barrel of this culvert was surveyed. The measuring wheel was zeroed on the east wall and the end of entry was the south end for each barrel. The cross section of this culvert is shown in Figure 3-17, and the entrance can be seen in Figure 3-18.

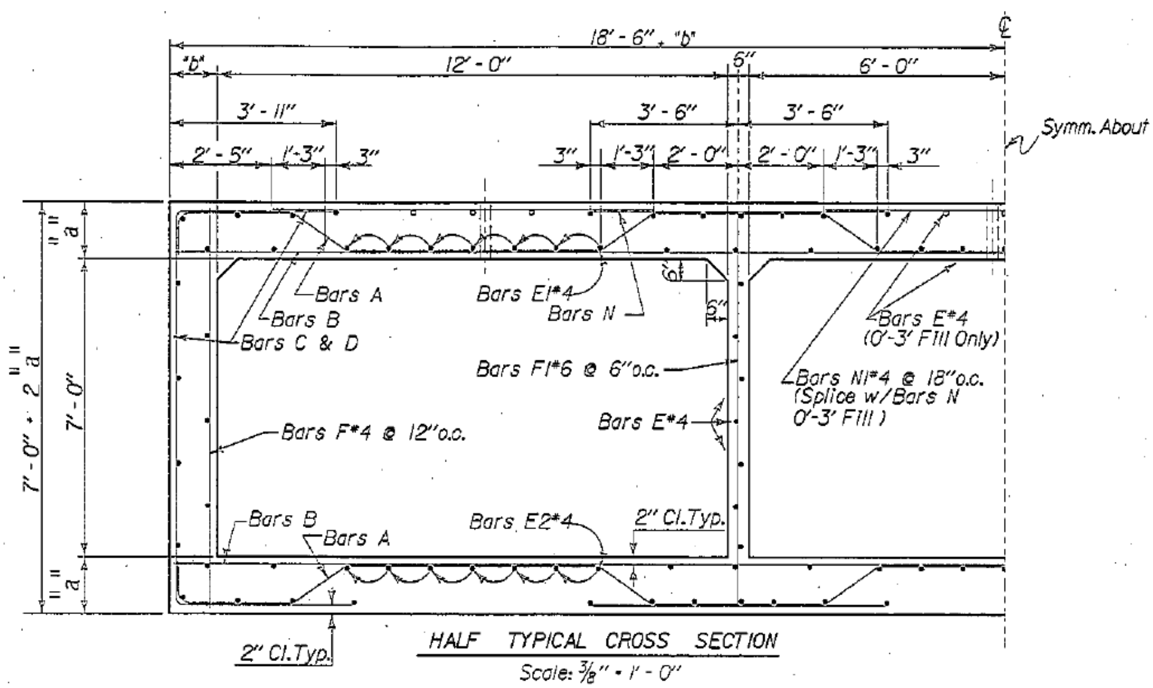


Figure 3-17: Centreville Culvert at 1808+98 Cross Section (ALDOT 2007)

Figure 3-18: Centreville Culvert at 1808+98 Entrance

This culvert had experienced cracking before the culvert was even backfilled. The widest crack observed in the eastern barrel was 0.02 in. (0.5 mm) wide. The widest crack observed in the center barrel was 0.03 in. (0.8 mm) wide. The widest crack observed in the western barrel was 0.02 in. (0.5 mm) wide. The widest joint openings were 0.013 in. (0.33 mm) in the center barrel were and 0.02 in. (0.5 mm) in the eastern and western barrels. Many of the wall cracks had been covered with mortar since the initial crack survey was performed by ALDOT (see Figure 3-19). The locations of these cracks were documented, and the widths of these cracks were taken from a previously performed ALDOT crack survey. The ALDOT survey is included in Appendix A. Cracking was also observed at the wingwall and culvert wall intersection as shown in Figure 3-20. Through cracks, some wider than 0.03 in. (0.8 mm), were observed on the outside of the ceiling of the culvert too as shown in Figure 3-21.

Figure 3-19: East Barrel Wall Crack Covered with Mortar in Centreville Culvert at 1808+98

Figure 3-20: Crack at Wingwall and Culvert Intersection in Centreville Culvert at 1808+98

Figure 3-21: Ceiling Crack on the Outside of Centreville Culvert at 1808+98

3.3.3 Corridor X

The culverts surveyed were located on Interstate 22 in Jefferson County. Corridor X Culvert at 4877+13 is approximately 0.5 mile (0.8 km) west of County Road 77. Corridor X Culvert at 4959+43 is approximately 1 mile (1.6 km) east of County Road 77. Corridor X Culvert at Exit 85 is located at the Exit 85-1 Mile sign on the westbound highway.

3.3.3.1 Corridor X Culvert at 4877+13

This crack survey was performed on September 16, 2010. The cross section of the culvert is shown in Figure 3-22. The southernmost barrel was chosen for the crack survey. The south wall was an exterior wall and the north wall was an interior wall. The measuring wheel was zeroed on the south wall and the end of entry was the east end.

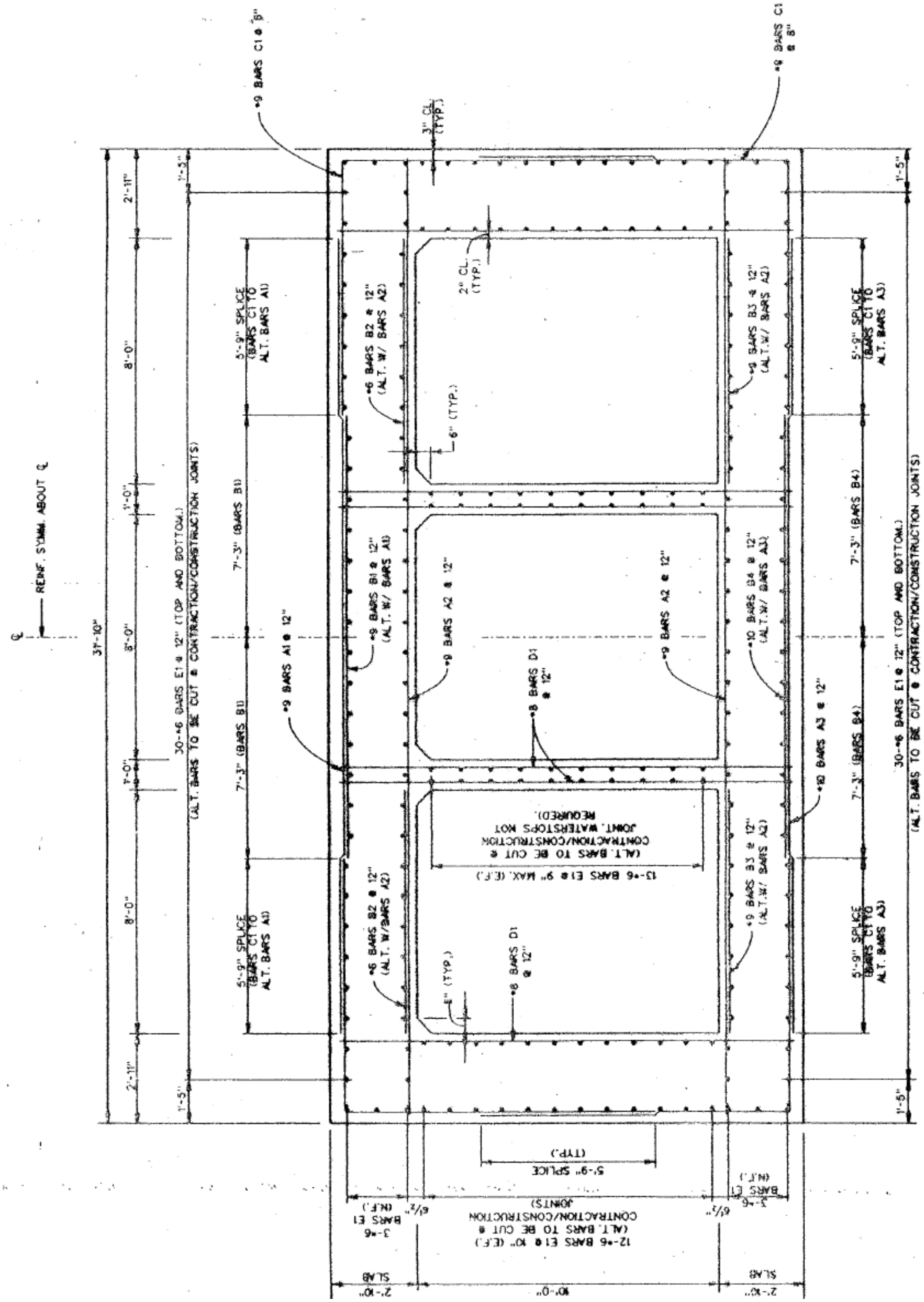


Figure 3-22: Corridor X Culvert at 4877+13 Cross Section (ALDOT 2000a)

Corridor X Culvert at 4877+13 was built with transverse contraction joints. The contraction joint detail from the plans is shown in Figure 3-23, and the average joint spacing used was 38 ft (12 m). However, the contraction joints used in the interior walls and ceiling did not have sealant in the joint groove as specified. The exceptions were the contraction joints in the interior walls at stations 39 ft 9 in. and 77 ft 0 in., which were built with sealant. The exterior walls and base contraction joints all had sealant, although the joint filler or sealant had come loose in many of the contraction joints in the base as shown in Figure 3-24. An exterior wall contraction joint is shown in Figure 3-25. It should be noted that 8 of the 23 transverse contraction joints in the walls had cracks adjacent to the joint, and 6 of the 8 cracks occurred in the exterior wall. These cracks ranged from hairline cracks to cracks as wide as 0.10 in. (2.5 mm). These cracks should not form next to a joint that allows movement; therefore, a question about whether the contraction joints in this culvert function correctly is raised. The cracks could be a result of the restraint provided by alternate bars being continuous through the contraction joint.

The ceiling of this culvert was too high to measure. Therefore, the ceiling crack widths were estimated relative to the crack widths measured in the walls. Also, the base was muddy and cracks were impossible to see. Because of this, joints in the base at stations 39 ft 9 in. and 77 ft 0 in. could not be observed. The widest crack observed was 0.10 in. (2.5 mm) wide and it was located at station 680 ft 10 in. in the exterior wall.

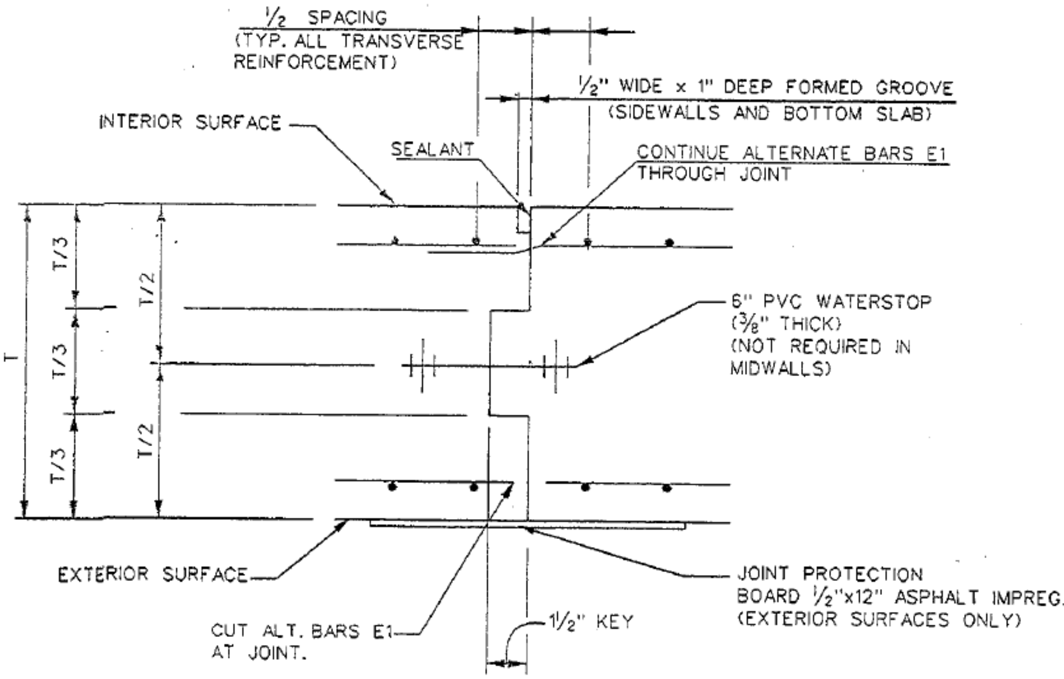


Figure 3-23: Transverse Contraction Joint Used in Corridor X Culvert at 4877+13 (ALDOT 2000a)

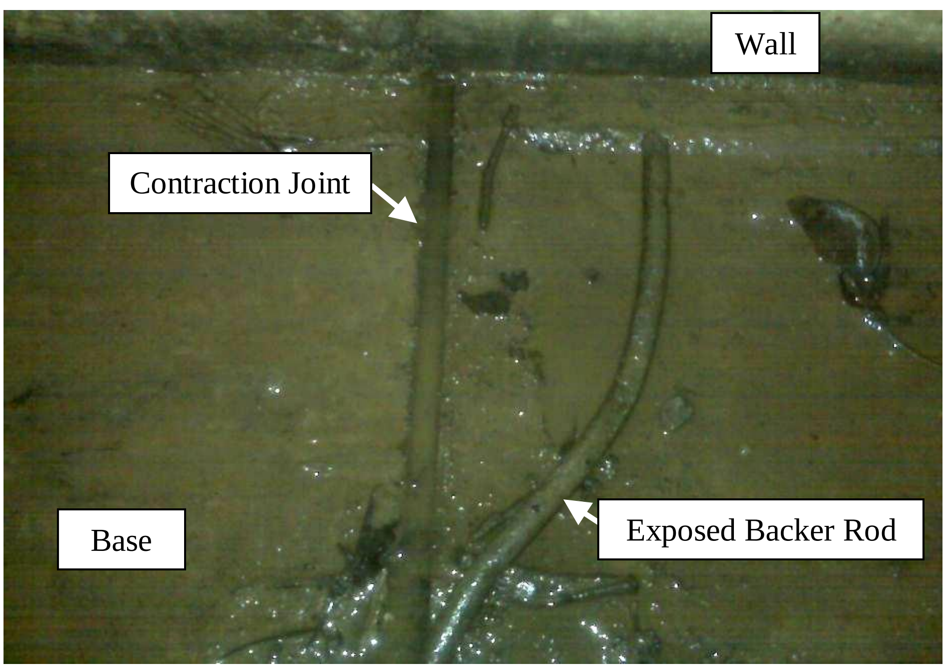


Figure 3-24: Transverse Contraction Joint in the Base of Corridor X Culvert at 4877+13 at Station 151 ft 8 in.

Figure 3-25: Transverse Contraction Joint in the Exterior Wall of Corridor X Culvert at 4877+13 at Station 341 ft 3 in.

3.3.3.2 Corridor X Culvert at 4959+43

This crack survey was performed on September 16, 2010. The cross section of Corridor X Culvert at 4959+43 is shown in Figure 3-26, and the entrance to the culvert is shown in Figure 3-27. The southernmost barrel was chosen for the crack survey. The south wall was an exterior wall and the north wall was an interior wall. The measuring wheel was started on the north wall and the end of entry was the west end.

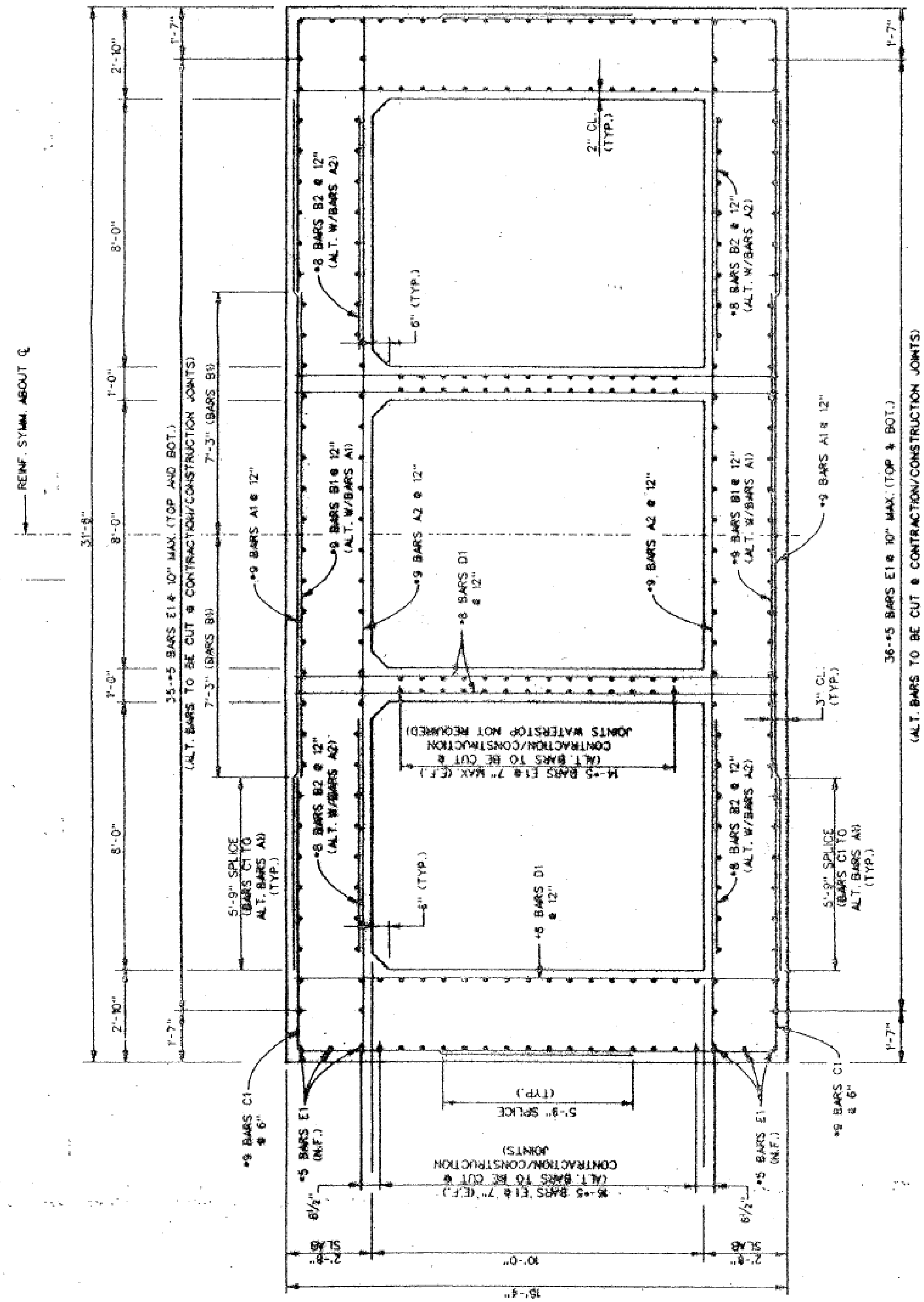


Figure 3-26: Corridor X Culvert at 4959+43 Cross Section (ALDOT 2000b)

Figure 3-27: Corridor X Culvert at 4959+43 Entrance and Wingwall Crack

The ceiling of this culvert was too high to measure. Therefore, the ceiling crack widths were estimated relative to the crack widths measured in the walls. Also, the base was muddy and made it difficult to find cracks. Because of this, joints and cracks in the base could not be observed in this culvert. The widest crack observed was 1/8 in. (3.2 mm) in the ceiling at station 334 ft 4 in. Cracks were also observed in the wingwall as seen in the left wingwall in Figure 3-28.

The transverse joint that was used in the construction of Corridor X Culvert at 4959+43 does not match the transverse contraction joint in the plans as shown in Figure 3-28. The average joint spacing used was 44 ft (13 m). The joint used in construction had no sealant, as can be seen in Figure 3-29. Some of the joints were open wide enough where they could be looked into with a flashlight. No reinforcement could be seen through the joint, and a key could not be seen either. The joints were still determined to be contraction joints because of the lack of continuous reinforcement.

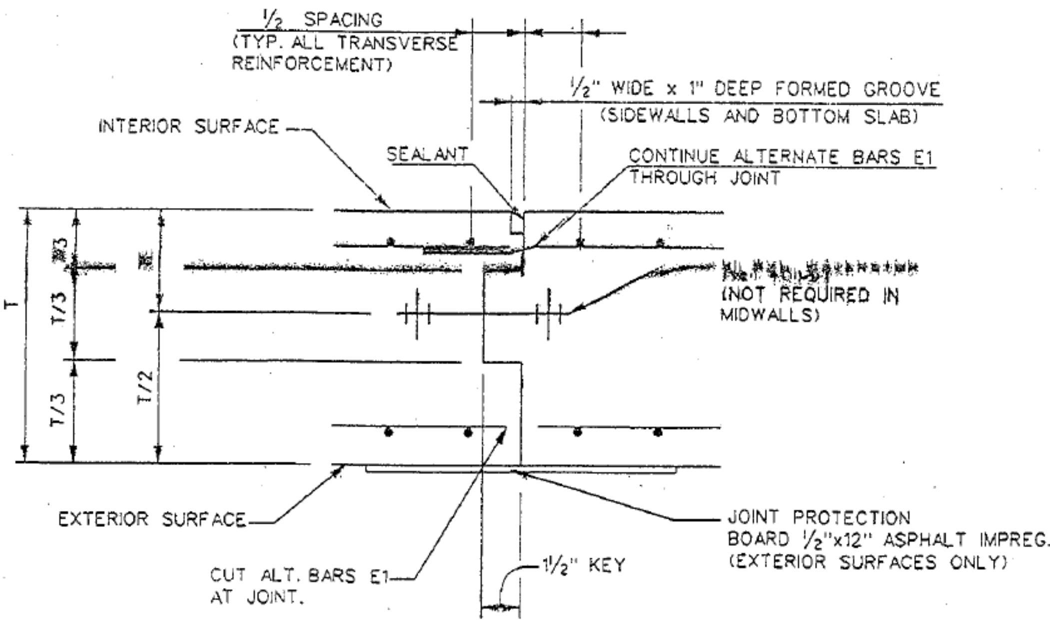


Figure 3-28: Transverse Contraction Joint used in Corridor X Culvert at 4959+43 (ALDOT 2000b)

Figure 3-29: Joint Opening Located in the wall of Corridor X Culvert at 4959+43

3.3.3.3 Corridor X Culvert at Exit 85

This crack survey was performed on June 12, 2012. Corridor X Culvert at Exit 85 was built with transverse construction joints that contained continuous longitudinal reinforcement in it and not contraction joints. The average joint spacing used was 46 ft (14 m). The entrance to Corridor X Culvert at Exit 85 is shown in Figure 3-30. The easternmost barrel was chosen for the crack survey. The east wall was an exterior wall and the west wall was an interior wall. The measuring wheel was zeroed on the east wall and the end of entry was the south end. The plans for this culvert could not be obtained.

Figure 3-30: Corridor X Culvert at Exit 85 Southern Entrance

Much of the base was under water and hard to see. Because of this, few joints and cracks in the base were observed in this culvert. The crack survey was stopped at station 965 ft 0 in. because of sediment build up in the culvert. The widest transverse crack observed was 0.05 in. (1 mm) wide in the east wall at stations 756 ft 3 in, 844 ft 5 in., and 931 ft 4 in. and also in the ceiling at station 756 ft 3 in. A transverse wall crack is shown in Figure 3-31. The largest joint opening was 0.10 in. (2.5 mm) in the east wall at station 775 ft 0 in and at station 822 ft 6 in. in the east wall and ceiling. A construction joint is shown in Figure 3-32. Longitudinal cracking was observed in the ceiling and at the chamfer.

Many of the transverse cracks and some of the construction joints had corrosion stains and/or deposits. The construction joints in the base appeared to be sealed; however, the joints in the walls and ceiling were not.

Figure 3-31: Transverse Wall Crack in Corridor X Culvert at Exit 85

Figure 3-32: Transverse Construction Joint in Corridor X Culvert at Exit 85 (Joint Patched with Mortar)

3.3.4 Dadeville

3.3.4.1 Dadeville Culvert at 45+31.55

A crack survey was performed on December 7, 2010. Dadeville Culvert at 45+31.55 was located south of Dadeville on SR-49 over Sougahatchee Creek, just south of CR-15. This culvert was built with transverse construction joints that contained continuous longitudinal reinforcement in it and not contraction joints. The average joint spacing used was 44 ft (13 m). The vee joint in Figure 1-3 was specified in the plans. The southernmost culvert was surveyed, and the end of entry was the west end. The measuring wheel was kept on the south wall. The interior wall was the north wall and the exterior was the south wall. The symmetric cross section of Dadeville Culvert at 45+31.55 is shown in Figure 3-33, and the entrance is shown in Figure 3-34.

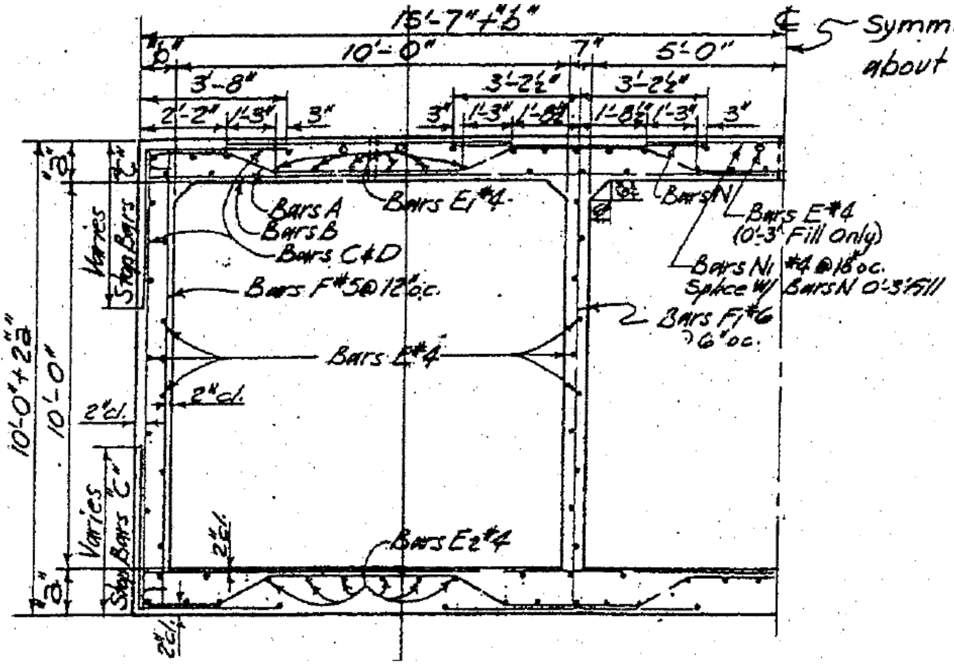


Figure 3-33: Dadeville Culvert at 45+31.55 Half-Typical Cross Section (ALDOT 1983)

Figure 3-34: Dadeville Culvert at 45+31.55 Entrance

The base could not be observed until after station 170 ft 0 in. because it was covered with mud and water. Also, the ceiling was too tall to be measured; therefore the crack widths were estimated.

The widest joint opening observed was 0.05 in. (1 mm) wide at station 222 ft 2 in. in the ceiling, exterior wall, and interior wall. The widest transverse crack was also 0.05 in. (1 mm) wide, and it was located at station 191 ft 8 in. in the exterior wall. A transverse crack detected in the wall of Culvert Dadeville 45+31.55 is shown in Figure 3-35. Other observations and measurements were also made throughout the survey. Many of the cracks had been repaired with epoxy, but new cracks had developed beside the epoxy-filled crack since that time. Also, there were wide cracks and distress where the wingwall connects to the culvert, as shown in Figure 3-36.

Figure 3-35: Transverse Wall Crack in Dadeville Culvert at 45+31.55

Figure 3-36: Dadeville Culvert at 45+31.55 Wingwall and Culvert Connection

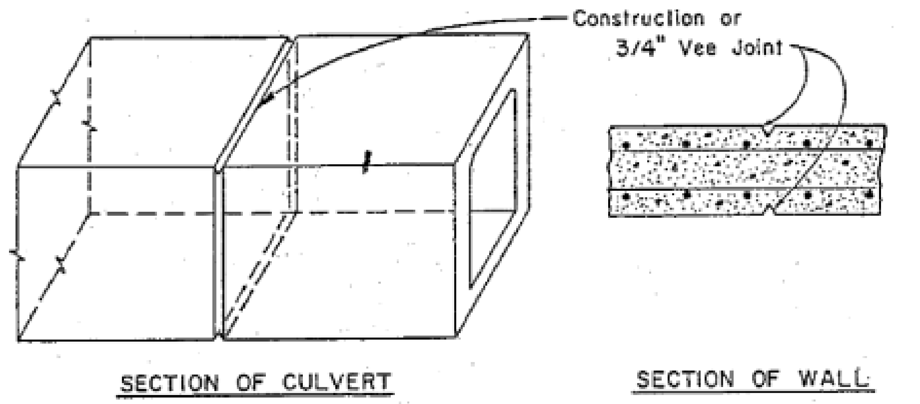
3.3.5 Dutton

3.3.5.1 Dutton Culvert at 548+23

A crack survey was performed on August 11, 2010. Dutton Culvert at 548+23 was built with transverse construction joints instead of contraction joints. The average joint spacing used was 46 ft (14 m). The transverse vee joint was specified in the plans, and it is shown in Figure 3-37. The measuring wheel was started in the middle to avoid snakes located on both walls, and it was moved to the north wall (oriented parallel to the ground) at station 322 ft 6 in. when the water became too deep. The end of entry was the east end. The entrance to the culvert can be seen in Figure 3-38, and the cross section of the culvert is shown in Figure 3-39.

After station 273 ft 7 in., the base of the culvert base could no longer be observed because it was underwater. Also, the survey was stopped at station 458 ft 5 in. because the water became too deep.

The widest transverse crack observed was 1/4 in. (6.4 mm) wide at station 110 ft 11 in. in both walls and the ceiling. The widest transverse joint opening observed was 3/8 in. (9.5 mm) wide at station 458 ft 3 in. in the south wall. Many of the transverse construction joints surveyed were tight or exhibited minimal movement. The transverse vee joint could be seen in the base at stations 90 ft 0 in. and 178 ft 5 in. Ceiling and wall thickness measurements were also taken during the survey. The ceiling was measured to be 11 in. (279 mm) deep at station 226 ft 7 in., 15.5 in. (394 mm) at station 410 ft 0 in., and 18.5 in. (470 mm) at station 457 ft 5 in. The north wall was measured to be 10.5 in. (267 mm) thick at station 232 ft 3 in. Figure 3-40 shows an example of a transverse crack observed in Dutton Culvert at 548+23. Longitudinal cracks that were 0.016 in. (0.41 mm) wide were observed in both walls.



NOTE

A ¾" Vee Joint is equivalent to and can be used instead of a Construction Joint for culvert construction. The term Joint shall refer to either and shall be determined by the project engineer.

No joint is required for culverts up to 60 ft long. Culverts 60 ft to 90 ft long require one joint, 90 ft to 135 ft two joints, and 135 to 175 ft three joints.

For culverts over 170 ft long place joints at approximate equal intervals of not less than 40 ft nor more than 55 ft. The joints shall be normal to the center-line of the culvert with longitudinal reinforcing extending through the joint. Use no key or expansion material in joints.

Figure 3-37: Transverse Vee Joint Used in Dutton Culvert at 548+23 (ALDOT 1980)

Figure 3-38: Dutton Culvert at 548+23 Entrance

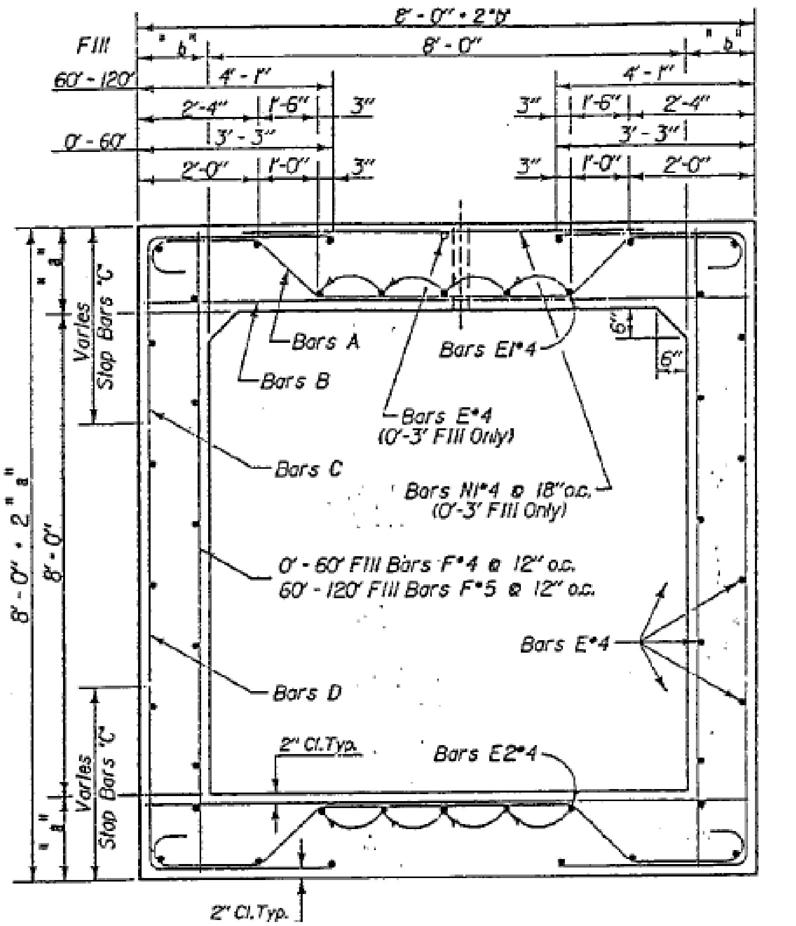


Figure 3-39: Dutton Culvert at 548+23 Cross Section (ALDOT 1988)

Figure 3-40: Transverse Crack in South Wall at Station 110 ft 11 in. of Dutton Culvert at 548+23 that is Greater than 1/4 in. (6.4 mm) Wide

3.3.6 Montgomery

3.3.6.1 Prattville Culvert on US-82

This crack survey was performed on May 5, 2011. Prattville Culvert on US-82 was located in Prattville, Alabama on US Highway 82 just past mile marker 143. This culvert was built with transverse construction joints that contained continuous longitudinal reinforcement and not contraction joints. The average joint spacing used was 30 ft (9.1 m). The eastern barrel of the culvert was surveyed. The measuring wheel was started on the east wall, and the end of entry was the north end. The entrance is shown in Figure 3-41. The eastern wall was the exterior wall and the western wall was the interior wall. The plans for this culvert could not be obtained.

The culvert had a major scour problem under the south half of the culvert. This caused significant settlement to occur, as can be seen in Figure 3-42. Cracks from station 51 ft 9 in. to the end of the culvert were documented but were noted as cracks that possibly could have been caused by the settlement. The widest crack observed was 5/16 in. (7.9 mm) wide at station 26 ft 10 in. in the exterior and interior wall. The widest joint opening was 0.25 in. (0.64 mm) wide at station 60 ft 0 in. in the ceiling, base, and both walls.

Figure 3-41: Entrance to Prattville Culvert on US-82

Figure 3-42: Settlement of Prattville Culvert on US-82

The concrete in the base of the culvert in the half that settled was severely deteriorated. This is illustrated below in Figure 3-43. The connection between the wingwalls and the culvert walls also had severely deteriorated. This is illustrated in Figure 3-44. Spalling was observed in the east wall at station 38 ft 0 in. Also, the construction joint at station 60 ft 0 in. (the point where the settlement started) had no reinforcement through it at all. A longitudinal crack was observed in the base at station 70 ft 4 in. The construction joints surveyed were open and showed movement of 5/16 in. (7.9 mm) at station 26 ft 10 in. and 0.25 in. (6.4 mm) at station 60 ft 0 in. Longitudinal cracks were observed in the section that had settled.

Figure 3-43: Base Deterioration in Prattville Culvert on US-82

Figure 3-44: Wingwall Connection to the Culvert in Prattville Culvert on US-82

3.3.6.2 I-85 North Culvert

This crack survey was performed on May 5, 2011. I-85 North Culvert was located on Interstate 85 North near exit 9. The measuring wheel was started on the west wall, and the end of entry was the north end. The entrance is shown in Figure 3-45. The interior wall was the west wall and the exterior wall was the east wall. Culvert extensions were added on to the original culvert at each end. The plans for this culvert could not be obtained, but transverse vee joints with continuous longitudinal reinforcement were found in this culvert. The average joint spacing used was 35 ft (11 m).

Figure 3-45: Entrance to I-85 North Culvert

There were three construction joint locations. The two outer construction joints were skewed and connected the outer culvert extensions to the original culvert. The middle construction joint was the vee joint that is shown in Figure 3-40. The southernmost construction joint was patched with mortar and the openings could not be measured. The northernmost construction joint had openings as wide as 0.06 in. (2 mm) and the middle construction joint had openings as wide as 0.025 in. (0.64 mm). The widest crack surveyed was 0.04 in (0.7 mm) wide at station 79 ft 6 in. in the exterior wall.

3.3.6.3 I-85 South Culvert

This crack survey was performed on May 5, 2011. I-85 South Culvert is located on Interstate 85 South near exit 6. The measuring wheel was started on the east wall, and the end of entry was the south end. The entrance is shown in Figure 3-46. The interior wall was the west wall and the exterior wall was the east wall. Culvert extensions were added at each end of the culvert. The plans for this culvert could not be obtained, but transverse vee joints with continuous longitudinal reinforcement were found in this culvert. The average joints spacing used was 41 ft (12 m).

Figure 3-46: Entrance to I-85 South culvert

There were three construction joint locations in the culvert. The two outer construction joints were skewed and connected the outer culvert extensions to the original culvert. The middle construction joint was a vee joint, shown in Figure 3-47 below. The southern most construction joint had openings as wide as 0.07 in. (2 mm), and the northernmost construction joint had openings as wide as 3/16 in. (4.8 mm). The middle construction joint had openings as wide as 0.075 in. (1.9 mm). The widest crack surveyed was 0.10 in. (2.5 mm) wide and it was observed in the east wall at station 75 ft 5 in.

Figure 3-47: Crack in the Vee Joint in the Wall in I-85 South Culvert

3.4 Evaluation of Culvert Condition Survey Data

3.4.1 Crack Widths and Crack Spacings

3.4.1.1 Procedure

The spacing between successive transverse cracks in the surveyed box culverts was calculated and documented for each wall, the base, and the ceiling. These data were used to make a frequency histogram of the crack spacing. See Appendix A for the crack width and spacing histograms of each culvert. Hairline cracks, patched cracks, cracks of which the width could not be measured, transverse contraction joints, and construction joints with continuous longitudinal reinforcement that were not tight were included in calculating the crack spacing. Plastic shrinkage cracks were not included because they are not through cracks.

The crack width data recorded was also used to make a frequency histogram of the crack widths for each wall, the base, and the ceiling of each culvert. Hairline cracks were assigned a width of 0.005 in. (0.1 mm). Construction joints with continuous longitudinal reinforcement that were not tight were included in the calculations. Plastic shrinkage cracks and cracks that could not be measured were not included either. Openings in joints that were meant to allow movement, such as contraction joints, were also not included.

3.4.1.2 Results and Discussion

The average crack widths and average crack spacing for each culvert surveyed are summarized in Figures 3-48 and 3-49. Prattville Culvert on US-82 was not included in the following data due to the distress being related to severe settlement issues. The settlement cracks were very wide and skewed the scale of the graphs. The dashed line in Figure 3-48 represents the ACI 224 (2001) crack width limit of 0.012 in. (0.30 mm) from Section 1.1.2. The V and C designations in Figure 3-51 represent culverts with transverse vee joints (transverse construction joints with continuous reinforcement through them) or transverse contraction joints respectively. Also, the 90th percentile crack widths for each culvert can be seen in Figure 3-50. The crack widths and spacing in the base of the culverts were not included because of the lack of available data.

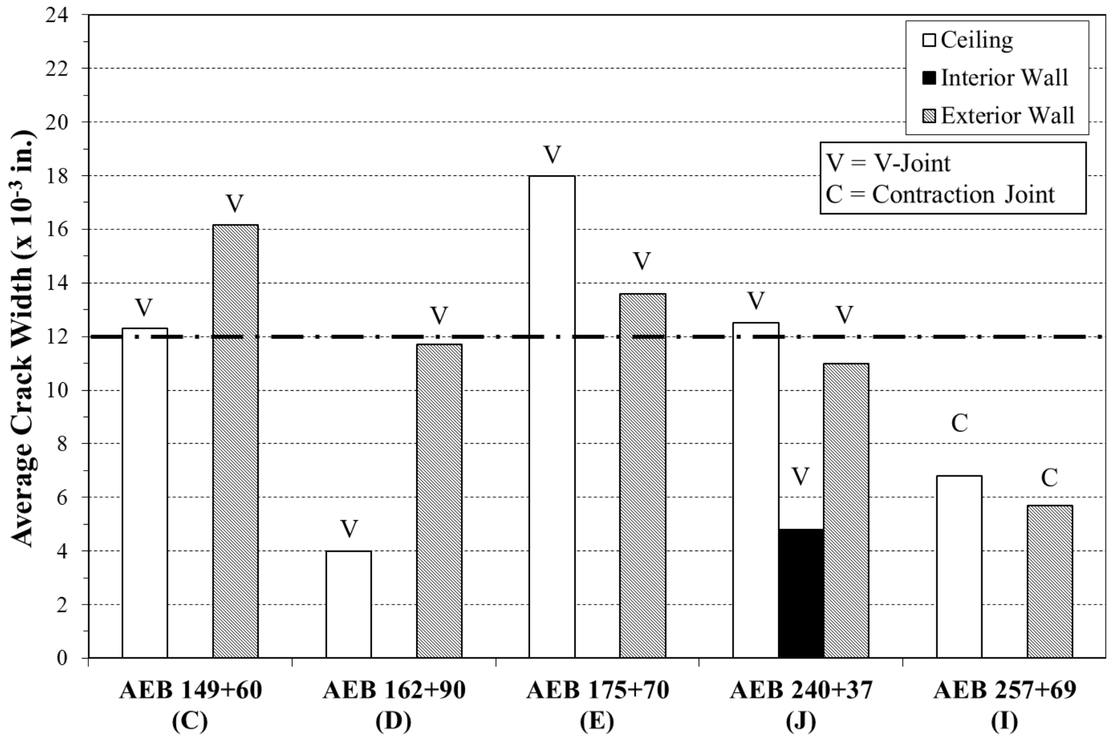


Figure 3-48a: Culvert Average Crack Widths

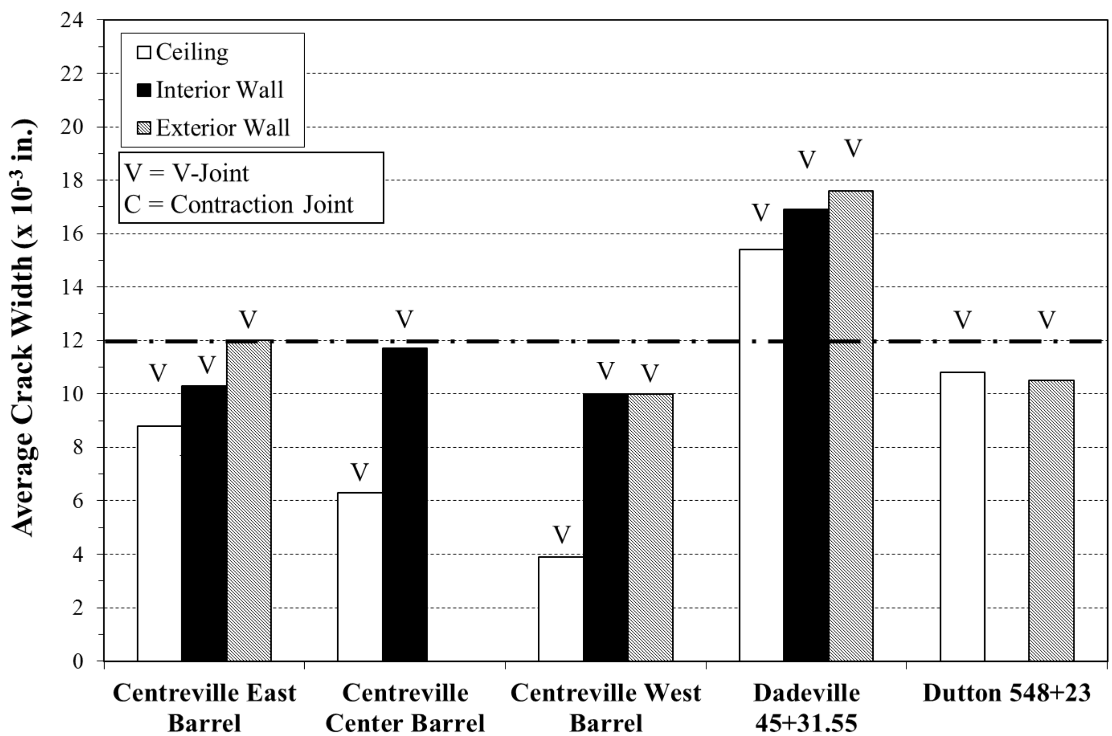


Figure 3-48b: Culvert Average Crack Widths

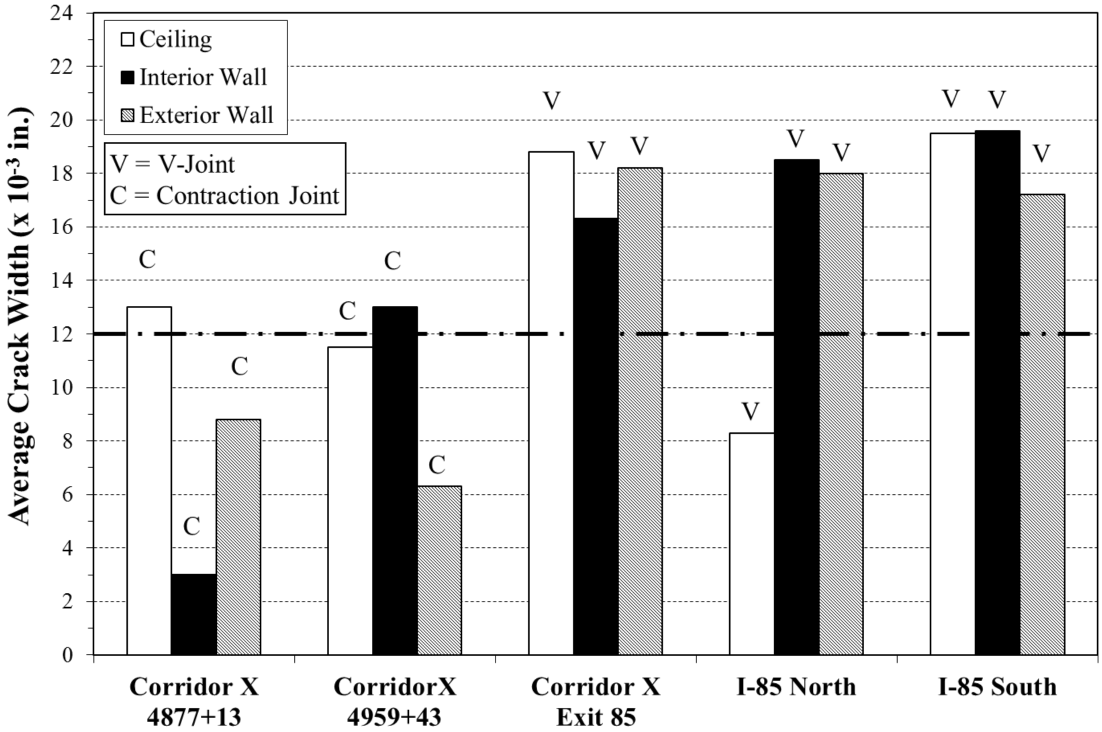


Figure 3-48c: Culvert Average Crack Widths

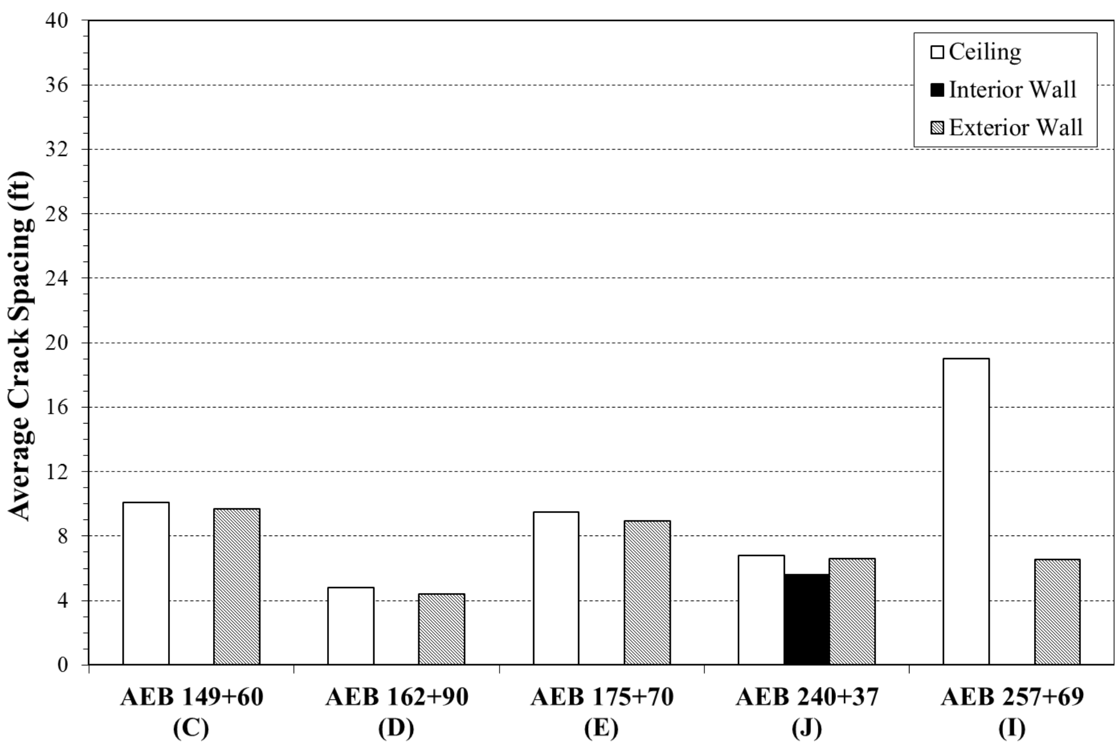


Figure 3-49a: Culvert Average Crack Spacing

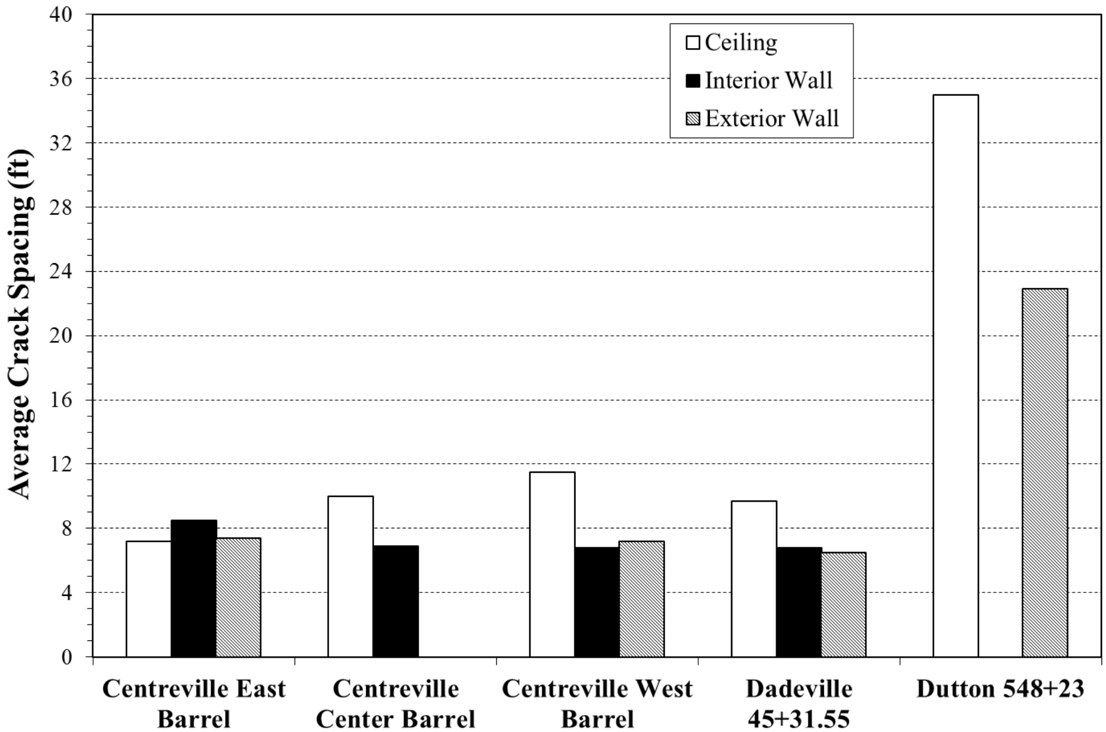


Figure 3-49b: Culvert Average Crack Spacing

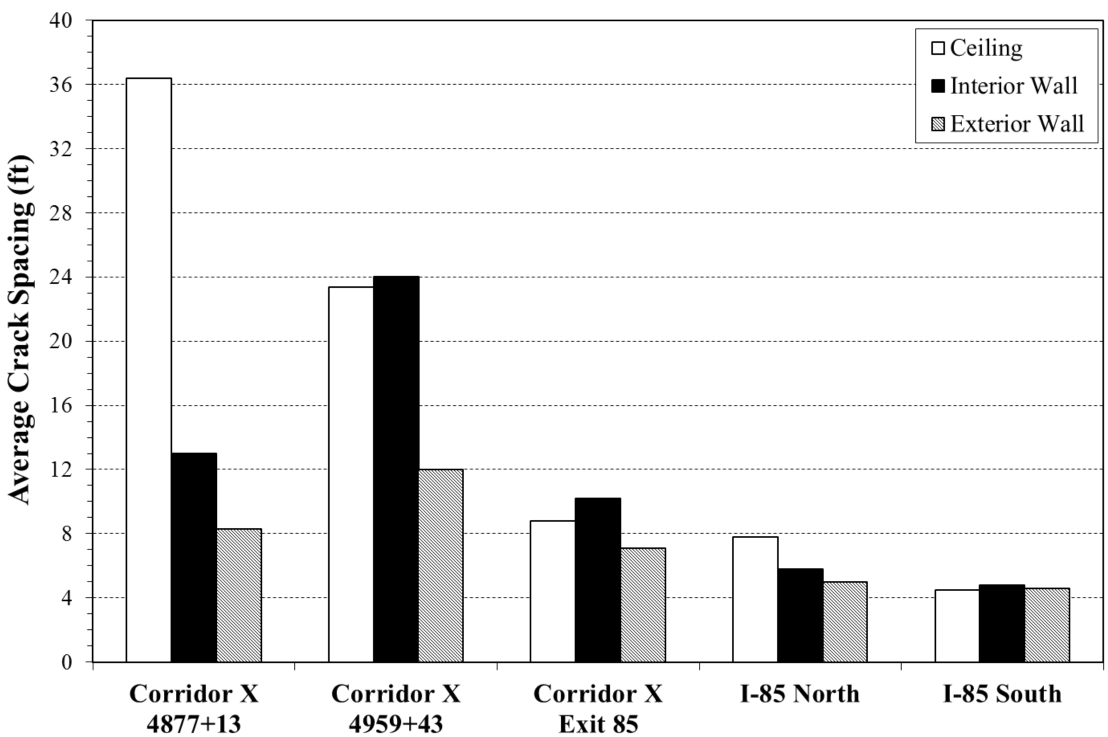


Figure 3-49c: Culvert Average Crack Spacing

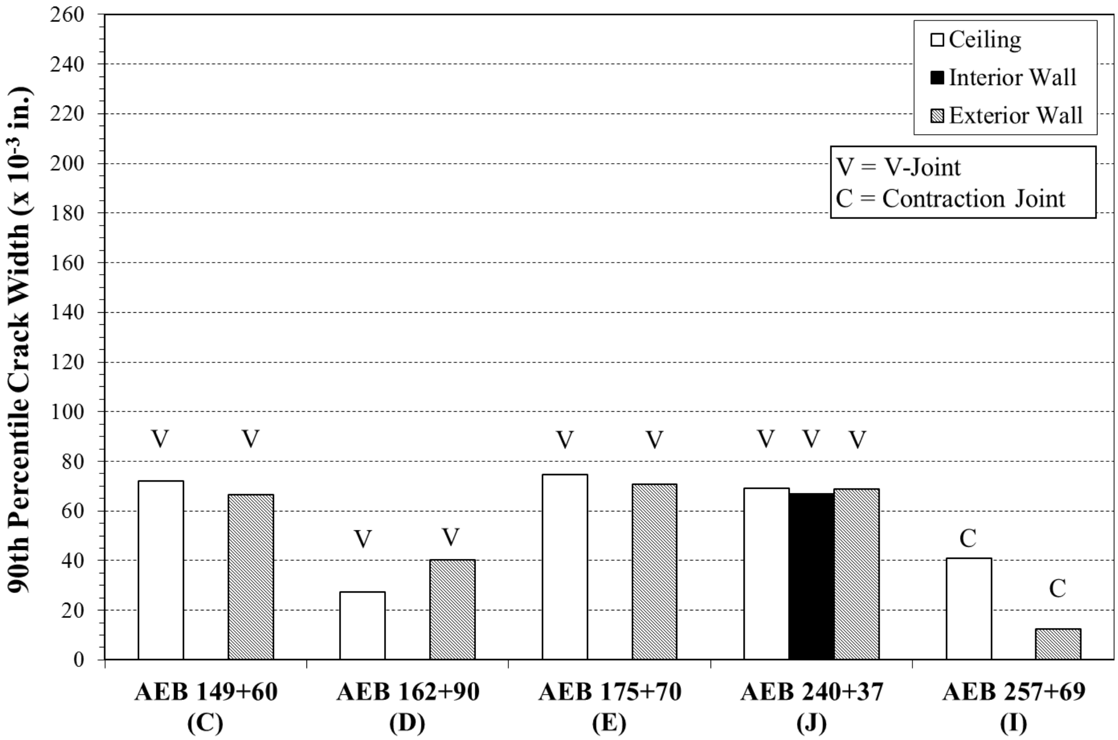


Figure 3-50a: Culvert 90th Percentile Crack Widths

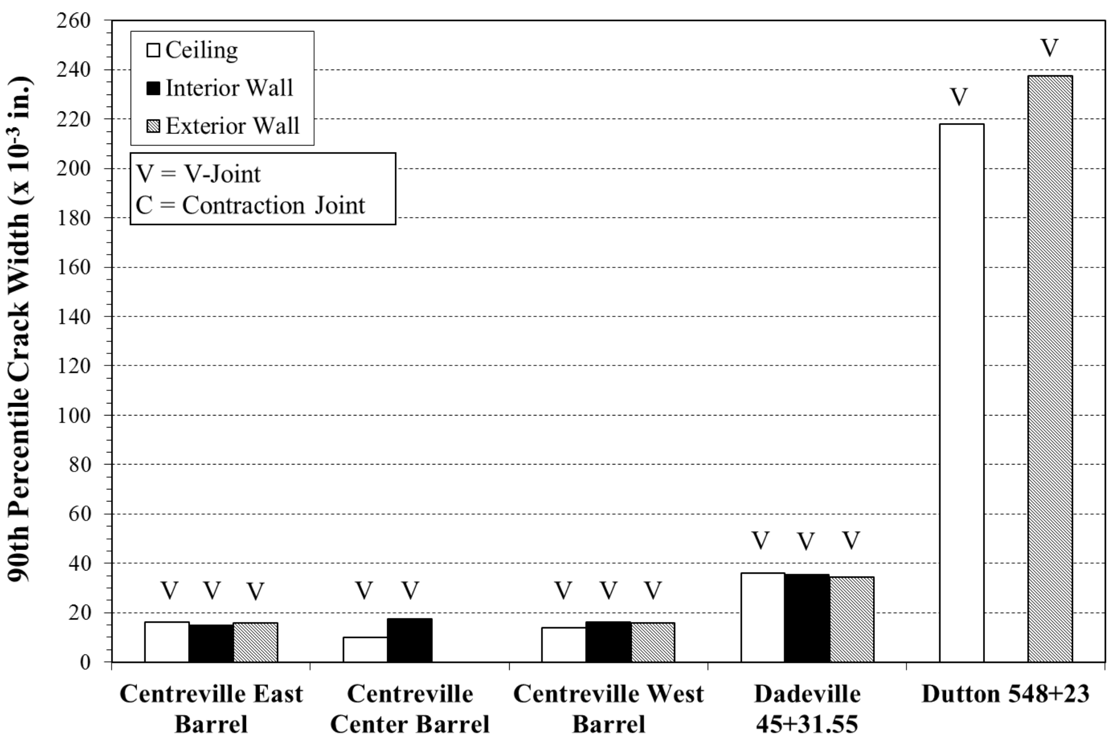


Figure 3-50b: Culvert 90th percentile Crack Widths

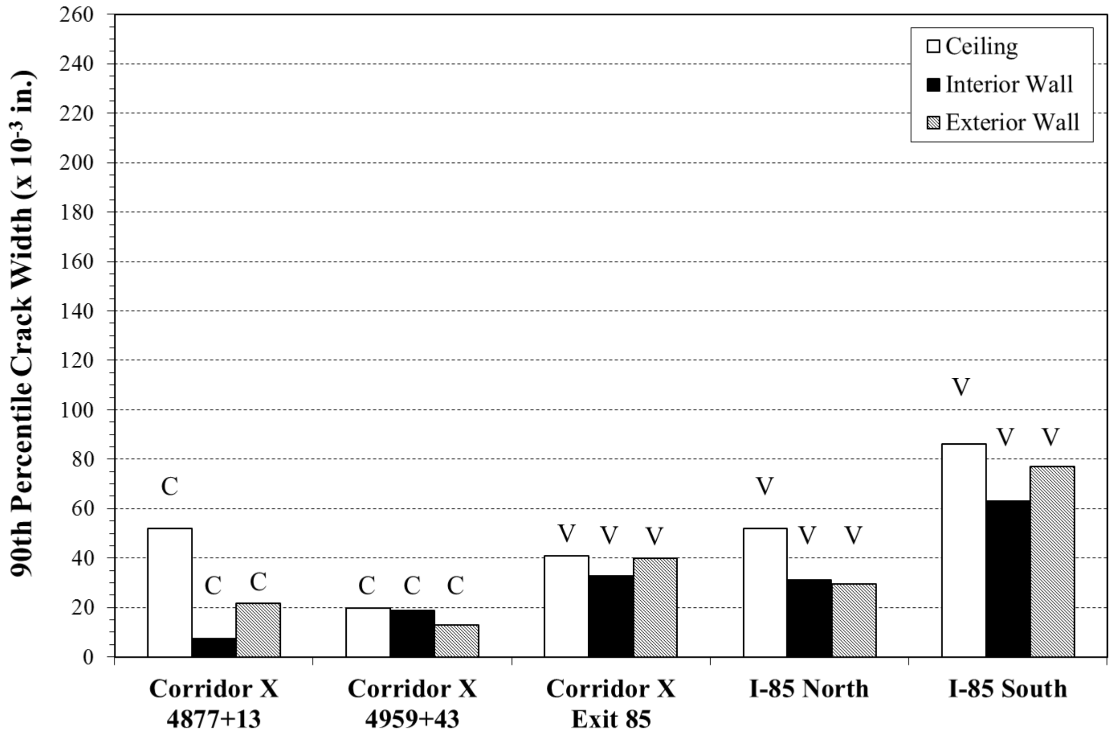


Figure 3-50c: Culvert 90th percentile Crack Widths

From the average crack width data, it can be concluded that the wide transverse cracks are not unique to the AEB project. Six of the eight culverts that were not located in the AEB project had a wall or ceiling with an average crack width that was greater than 0.012 in. (0.30 mm).

It can also be concluded that using transverse contraction joints in CIP reinforced concrete box culverts reduced the crack widths experienced. AEB Culvert at 257+69 was the only culvert in the AEB project that had transverse contraction joints. This culvert's average *wall* crack width was 0.006 in. (0.1 mm) as compared to an average of 0.011 in. (0.28 mm) for the other culverts surveyed in this project. Its average *ceiling* crack width was 0.007 in. (0.1 mm) as compared to 0.012 in. (0.30 mm) for the other culverts surveyed in this project. Therefore, using contraction joints *reduced* average crack widths by 48% in the *walls* and 43% in the *ceiling* in the AEB project. Similar results were found when comparing the 90th percentile crack widths of the culverts in the AEB project. The 90th percentile *wall* crack width of AEB Culvert at 257+69 was 0.012 in. (0.30 mm) as compared to 0.063 in. (1.6 mm) for the other culverts surveyed in this project. Its 90th percentile *ceiling* crack width was 0.041 in. (1.0 mm) as compared to 0.061 in. (1.6 mm) for the other culverts surveyed in this project. Therefore, using contraction joints *reduced* the 90th percentile crack widths by 81% in the *walls* and 33% in the *ceiling* in the AEB project. Two of the three culverts from the Corridor X project had transverse contraction joints. These two culverts, Corridor X Culvert at 4877+13 and Corridor X Culvert at 4959+43, had much smaller average crack widths than the culvert that did not have transverse contraction joints, Corridor X Culvert at Exit 85. The average *wall* crack width of the Corridor X culverts with contraction joints was 0.008 in. (0.2 mm) as compared to an average of 0.017 in. (0.43 mm) for the other culvert surveyed in this project. The average *ceiling* crack width was 0.012 in. (0.30 mm) for the culverts with contraction joints and 0.019 in. (0.48 mm) for the other culvert surveyed in this project. Therefore, using contraction joints *reduced* average crack widths by 54% in the *walls* and 36% in the *ceiling* in the Corridor X project. Corridor X Culvert at 4877+13 and Corridor X Culvert at 4959+43 also had much higher crack spacings than did Culvert X Exit 85. This implies that the culverts with transverse contraction joints had fewer, narrower cracks than the culvert without transverse contraction joints did. Mixed results were found when comparing the 90th percentile crack widths of Corridor X culverts. The two culverts with contraction joints had an average 90th percentile *wall* crack width of 0.012 in. (0.30 mm) as compared to 0.063 in. (1.6 mm) for the other culvert surveyed. This means that using contraction joints caused a *reduction* of 81% in the 90th percentile *wall* crack widths in the Corridor X project. However, when comparing the 90th percentile *ceiling* crack widths, the two culverts with contraction joints had an average 90th percentile crack spacing of 0.052 in. (1.3 mm) as compared to 0.041 in. (1.0 mm) for the other culvert surveyed. This means that using contraction joints caused a 27% *increase* in the 90th percentile *ceiling* crack widths in the Corridor X project, and makes the effect of contraction joints on the 90th percentile crack spacing inconclusive.

Examining the effects of contraction joints on average crack spacing provided inconclusive results. The culvert in the AEB project with contraction joints had average ceiling and wall crack spacings of 6.6 ft (2.0 m) and 19 ft (5.8 m) respectively. The culverts in the same project without contraction joints had average wall and ceiling crack spacings of 7.1 ft (2.2 m) and 7.8 ft (2.4 m) respectively. Therefore, using contraction joints *decreased* the *wall* crack spacing by 7% and *increased* the *ceiling* crack spacing by 144% when compared to the other culverts in the AEB project. In the Corridor X project, the culverts with contraction joints had average wall and ceiling crack spacings of 14.3 ft (4.36 m) and 36.4 ft (11.1 m) respectively. The culvert in this project without contraction joints had an average wall and ceiling crack spacing of 8.7 ft (2.6 m) and 8.8 ft (2.7 m) respectively. Using contraction joints *increased* the average *wall* crack spacing by 65% and *increased* the average *ceiling* crack spacing by 314% when compared to the other culvert in the Corridor X project. Because of the discrepancy between the Corridor X and AEB project findings, there were no conclusive relationships found between the average crack spacing and using contraction joints.

AEB Culvert at 162+90, Centreville Culvert at 1808+98, and Dutton Culvert at 548+23 were the only culverts with transverse construction joints with continuous longitudinal reinforcement through them that did not have a wall or ceiling with an average crack width greater than 0.012 in. (0.30 mm). It is not certain why these culverts performed better than other culverts with transverse construction joints and continuous reinforcement did not. It is possible that these culverts were primarily constructed during winter months, which would mostly negate all thermal stress development. Dutton Culvert at 548+23 also had a 90^{th} percentile crack width that was much greater than the other culverts surveyed. It had very few cracks and the majority of them were narrow; however, it had cracks at two locations that were greater than 1/4 in. (6.4 mm) wide. These very wide cracks accompanied with the small number of cracks found are the reason for the extreme 90^{th} percentile crack width value.

The majority of the culverts surveyed had an average ceiling and wall crack spacing less than 13 ft (4.0 m). The only culverts that did not were AEB Culvert at 257+69, Corridor X Culvert at 4877+13, Corridor X Culvert at 4959+43, and Dutton Culvert at 548+23. Corridor X Culvert at 4877+13 and Corridor X Culvert at 4959+43 are discussed above. AEB Culvert at 257+69 is mentioned above as well. As in the preceding paragraph, the large average crack spacings in Dutton Culvert at 548+23 could be explained by the possibility of it being placed in the winter.

While many of the culverts surveyed demonstrated adequate crack performance, all of the culverts still had cracks wider than 0.012 in (0.30 mm). There was still room for the crack performance to be improved. The majority of the culverts had transverse joints spaced at approximately 50 ft (15 m). Decreasing the joint spacing would be a way to improve the crack performance.

The typical distress observed at these sites was transverse through cracks in the walls and ceiling. Few base cracks were observed due to the view of the base usually being obstructed. Minor efflorescence and corrosion were observed at many locations during condition surveys, but the degree of distress associated with these were minimal.

Some longitudinal cracking was observed between the celling and the wall, but these cracks were expected. Longitudinal cracks are expected in the ceiling and base of CIP box culverts at the locations shown in Figure 3-51. These are the locations of maximum bending moment form the downward soil loads on the culvert. The maximum positive bending moment will be located near the middle of the ceiling and base where the tension fiber will be on the interior face. The locations of maximum negative moment will be near the supports where the tension fiber will be on the exterior face. Longitudinal cracks are generally not expected in the walls due to the compressive forces negating the bending moments from the horizontal soil loads. The longitudinal chamfer cracks observed can be attributed to stress concentration at a change in the geometry of the culvert cross section. The longitudinal cracks observed ranged for hairline to 0.016 in. (0.41 mm) wide. These cracks were not considered a major concern due to the infrequency of their occurrence.

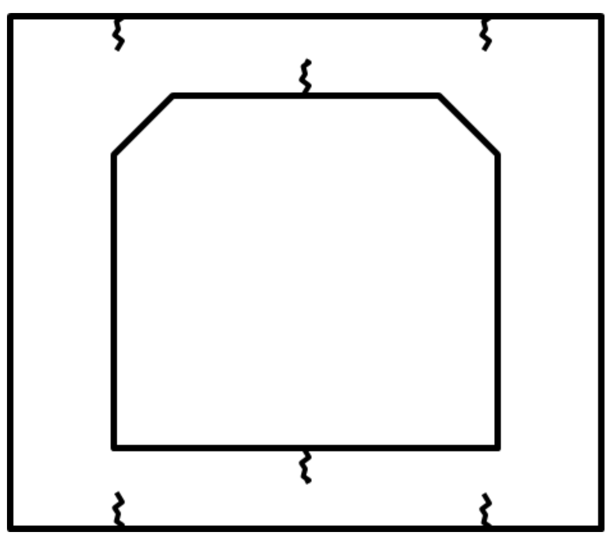


Figure 3-51: Expected Locations of Longitudinal Cracks in Box Culverts

Wingwall cracking was observed in several of the culverts surveyed. Distress was observed at wingwall culvert connections as well as in the wingwall itself. The distress was severe enough to where it should be addressed by redesigning the wingwall and culvert intersection and by adding joints to alleviate stresses.

From the culvert crack condition surveys and the data collected, the stress development due to the restraint of thermal and drying shrinkage appears to be the main cause of the wide transverse cracking observed in the culvert barrels. Drying shrinkage is inevitable, because the culverts are exposed to relative humidity conditions of less than 100% (Hansen and Almudaiheem 1987). The culverts are also exposed to hot summer and cold winter temperatures over time that make thermal volume changes unavoidable (Schindler 2002). The restraint provide by the stiff base along with the combined effects of thermal and drying shrinkage cause tensile stresses to rise in the wall and ceiling of the culverts. The tensile stresses rise until the tensile strength of the concrete is reached and through cracks form. Refer to Figure 2-19 for an illustration of this cracking mechanism. These transverse through cracks indicate that the distress is caused by restrained thermal shrinkage (Bernander 1998). Cracks may also form in the base, due to the restraint provided by the granular subbase; however, these cracks occur less frequently, because the restraint provided by the subbase is much less than the restraint the base provides for the walls and ceiling. The fact that using contraction joints in the AEB and Corridor X projects helped to limit these transverse crack width also suggest that restrained thermal and drying shrinkage are a major cause. Refer to Figure 2-19 for an illustration of how contraction joints relieve tensile stresses in walls and mitigate cracking. The generally systematic occurrence of cracks throughout the culverts – on average every 9.5 ft (2.9 m) – imply that settlement is not the cause of the transverse cracks. There would be localized distress if settlement was the cause of the transverse cracking. The through cracks indicate that plastic shrinkage is not the main causes of distress, because they tend to be shallow cracks (ACI 224 2007).

The main cause of the distress experienced in the wingwalls is also thought to be restrained thermal and drying shrinkage of the concrete. The rigid wingwall footings can provide restraint in long wingwalls that cause tensile stresses to rise when the wingwall contracts due to thermal and drying shrinkage. There is no contraction joint in the middle of the wingwalls to alleviate the stresses so cracking occurs. The distress in AEB Culvert at 240+37 (see Figure 3-12) is an example of this. Refer to Figure 2-19 for an illustration of this cracking mechanism. Distress at the intersection of the culvert and wingwall can also be explained by restrained drying shrinkage. The distress experienced in Dadeville Culvert at 45+31.55 (see Figure 3-36) is an example of this distress. The concrete volumetric changes in the long culvert are restrained by the wingwalls that have rigid footings preventing movement of the wall. This causes tensile stresses to rise, which are magnified by the change in angle where the culvert and wingwall interact, and cracking to occur. This concept is illustrated in Figure 3-52. However, settlement of the wingwall foundation, as suggested by ALDOT, is also a very possible cause. The wingwall distress is localized in some culverts at the intersection of the culvert and wingwall (see Figure 3-36). The cracking there could be a result of the wingwall rotating when one end of the wingwall foundation settles. The cracking in the wingwall itself (see Figure 3-12) could possibly be explained by foundation settlement under the middle of the wingwall. These examples of wingwall settlement are illustrated in Figure 3-53.

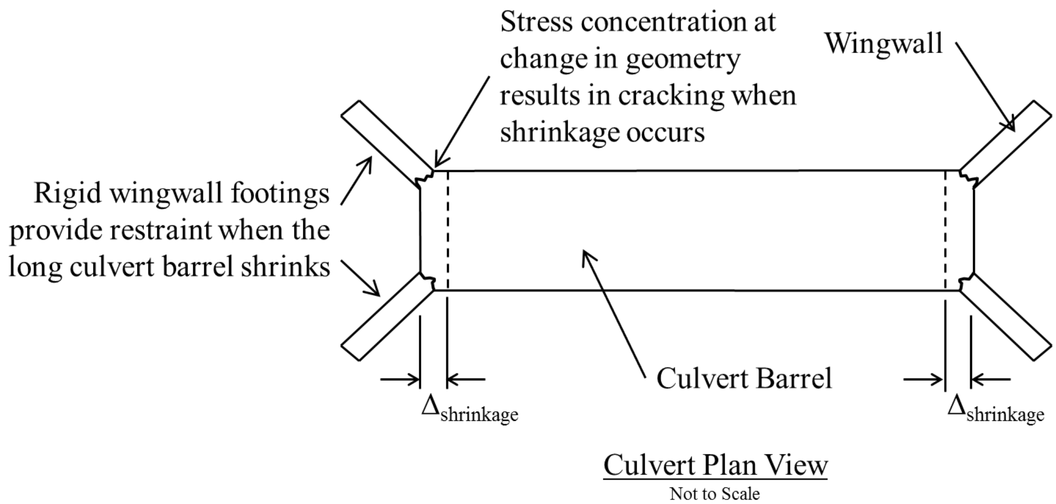


Figure 3-52: Cracking at the Wingwall and Culvert Intersection Illustration

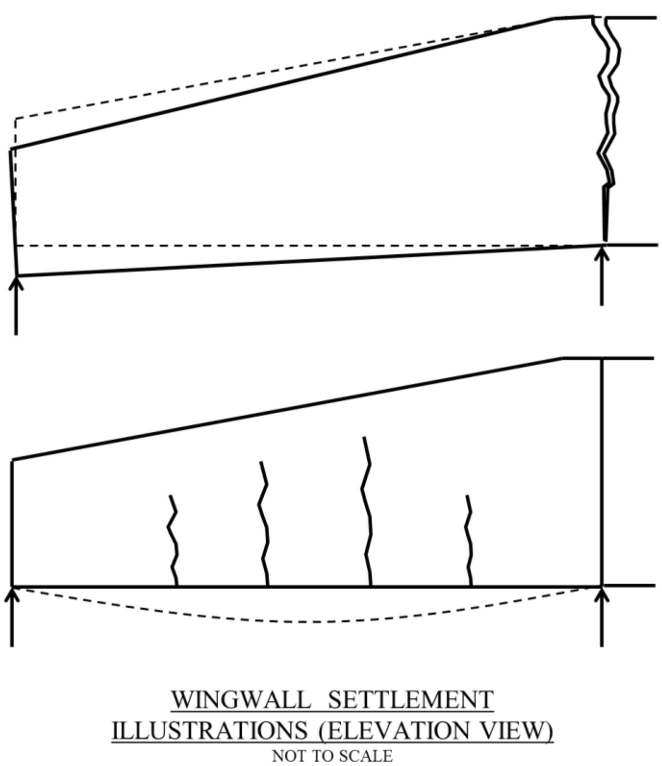


Figure 3-53: Wingwall Foundation Settlement Cracking Illustrations

The transverse through cracks found in Centreville Culvert at 1808+98 best illustrate the reasoning behind why restrained thermal and drying shrinkage is the most likely cause of distress at the culvert and wingwall interaction. The through cracks occurred before the culvert was even backfilled, which indicates that settlement is not a likely cause of cracking. The wingwall and culvert were still exposed to air on all sides, and the transverse vee/construction joints and very stiff base provided significant restraint. The wingwall crack observed (see Figure 3-20) was at a point of stress concentration (the change in geometry at intersection of the culvert and wingwall) and the stress resulting from restrained concrete volumetric change is a very likely cause of this distress.

3.5 Summary and Conclusions

In this chapter, the procedure for the box culvert condition surveys is outlined. The location, geometry, length, type of joint used, widest crack, and unique distress experienced are presented for each of the 14 culverts surveyed. The average crack width, average crack spacing, and 90th percentile crack width are also presented for the culverts.

The typical distress observed was transverse through cracking in the walls and ceiling of the culverts. Few transverse base cracks were observed, due to them being covered with mud or under water, but they tended to be wider than wall and ceiling cracks on average. Wingwall cracking was also observed where culvert and wingwall intersect as well as in the wingwall itself. The distress was severe enough to where wingwall joints should be added to reduce stresses, and the intersection of the culvert and wingwall should be redesigned.

Four key findings were drawn from the data presented in this chapter. The first is that 6 of the 8 culverts surveyed that were not located in the AEB project had a wall or ceiling with an average crack width greater than 0.012 in (0.30 mm). This is cause for concern and shows that the distress experienced is not unique to the AEB project. The second is that using transverse contraction joints in the culverts helped in mitigating cracking. When comparing culverts in the AEB and Corridor X projects, the culverts at each location that had transverse contraction joints had smaller average crack widths when compared to the culverts in the same location that did not have them. Relationships between contraction joints and the 90th percentile crack width or average crack spacing were inconclusive. The third finding is that the combined effect of restrained thermal and drying shrinkage is the most likely cause of cracking. The temperature and relative humidity cycles the culverts experienced over time make drying and thermal shrinkage inevitable (Hansen and Almudaiheem 1987; Schindler 2002). The significant restraint provided by the base, accompanied by drying and thermal shrinkage, causes tensile stresses to rise in the walls and ceiling of the culvert until through cracks form. Thermal and drying shrinkage being the main cause of distress is supported by the contraction joints helping to mitigate cracking in the AEB and Corridor X projects, and by the cracking occurring in Centreville Culvert at 1808+98 before it was backfilled. The fourth finding was that the wingwall cracking observed is most likely caused by restrained thermal and drying shrinkage also. The cracks found in the wingwall and at the intersection of the wingwall and culvert can be explained by the rigid wingwall footings providing restraint to thermal and drying shrinkage volume changes in the concrete. However, as ALDOT suggested, it is reasonable to assume that foundation settlement may also be a cause.

Chapter 4

Temperature and Shrinkage Reinforcement

4.1 Introduction

Slabs and walls may not have large bending moments at specific locations. These slabs and walls have small (or possibly minimum) amounts of flexural reinforcement at these locations, which can lead to very wide cracks due to shrinkage of the concrete if they are significantly restrained. These cracks are referred to as direct tension cracks, because they are caused by direct tension stresses in the concrete as opposed to flexural tension stresses. Significant amounts of reinforcement can be needed to control direct tension cracking. (Gilbert 1992)

The small amount of reinforcement in these structures with low moments is typically referred to as temperature and shrinkage reinforcement. It works by distributing the shrinkage strains over the reinforcement so that several small cracks form rather than a few detrimental wide cracks. (ACI 224 2001)

4.2 Direct Tension Cracking

The work in this chapter is based on an article from Gilbert (1992), and modifications made to Gilbert's work are clearly documented. Restraint prevents or resists volumetric changes in the concrete when shrinkage occurs. An axial tensile restraining force *N(t)* rises in the concrete over time due to the member being restrained in some way. When the stress from *N(t)* equals the direct tensile strength of the concrete, f_t, direct tension cracking occurs. Direct tensile cracks are through cracks, in that they extend the full depth of the member. (Gilbert 1992)

After cracking, the crack width and *N(t)* are mostly a function of the quantity of bonded reinforcement across the formed crack. Other factors that affect the crack width are the spacing and size of the reinforcement bars, the quality of the concrete, and the effectiveness of the bond between the concrete and the reinforcement. (Gilbert 1992)

4.2.1 First Cracking in Fully Restrained Concrete

When restrained, reinforced concrete shrinks, it will remain uncracked until the tensile stress in the concrete reaches its tensile strength, $f_{t.}$ This is illustrated in Figure 4-1a. *N(t)* will drop to a value of N_{cr} instantly after the first crack forms. This concept is illustrated in Figure 4-1b. The reinforcement carries all of N_{cr} at the crack location because the tensile strength of the concrete at the crack is zero. The tensile stress in the concrete adjacent to the crack (referred to as Region 2 as shown in Figure 4-1c) changes from zero at the crack until it reaches a constant value σ_{c1}, which is less than the tensile strength of the concrete, at a distance s_o away from the crack (referred to as Region 1 as shown in Figure 4-1c). The reinforcement stress also varies. At the crack, the steel is at its peak tensile stress σ_{s2}, and then it decreases to a constant compressive stress σ_{s1} at a distance s_o away from the crack. The steel and concrete stresses in Region 2 are assumed to vary according to a parabolic distribution as shown in Figures 4-1c and 4-1d. The stress in the steel and concrete in Region 1 are no longer affected by the crack as shown in Figures 4-1 c and 4-1d. (Gilbert 1992)

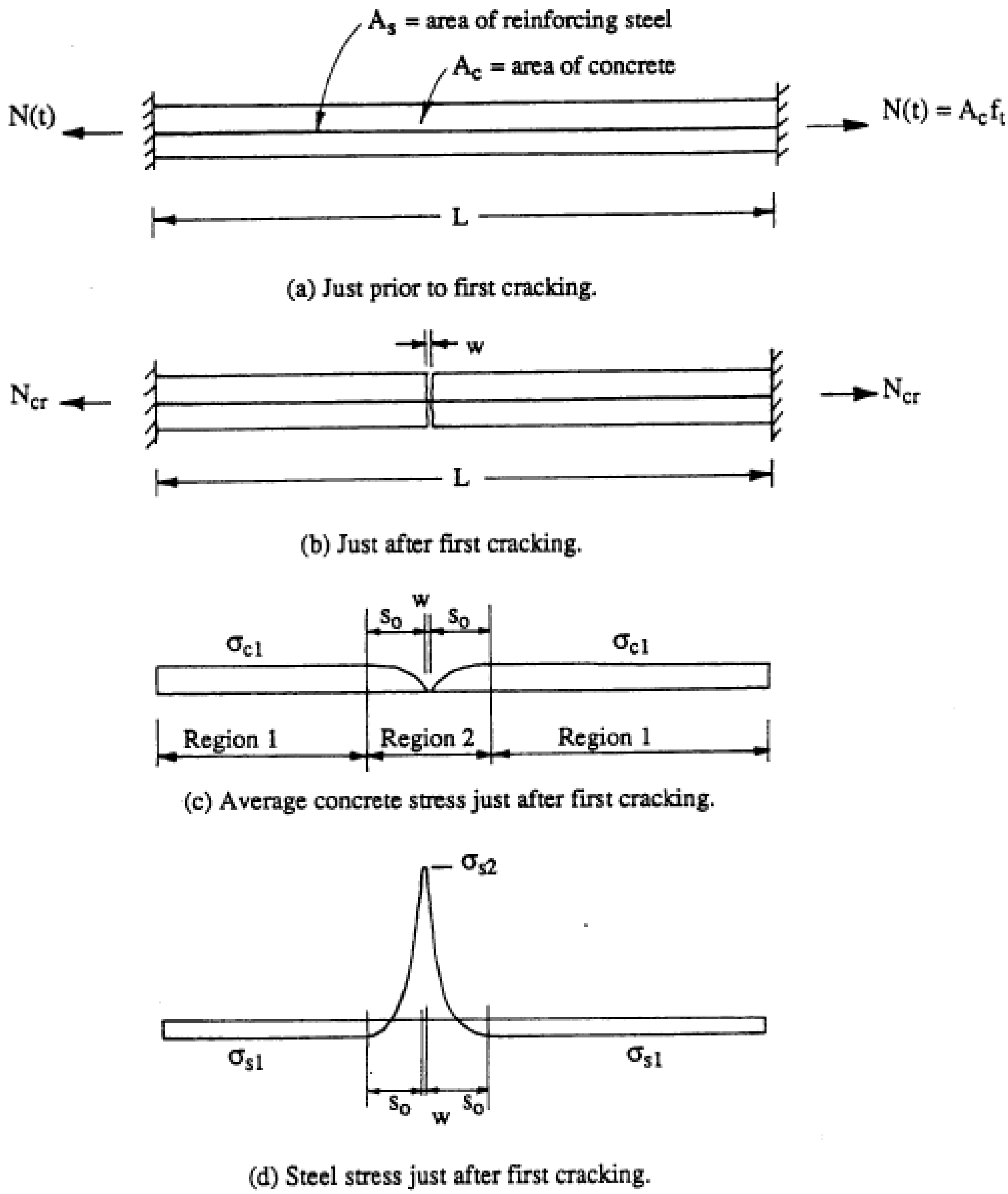


Figure 4-1: First Cracking Illustration (Gilbert 1992)

4.2.2 Final Cracking in Fully Restrained Concrete

The concrete is less restrained after initial cracking occurs. This is because the crack width increases as the concrete continues to shrink as shown in Figure 4-2a. Continued shrinkage causes *N(t)* to once again rise until a second crack forms. As the concrete continues to shrink over time, more cracks may form. However, the amount of shrinkage required to produce a new crack rises due to the member losing stiffness with each new crack. A final crack pattern will eventually be established from this process, and this typically occurs when most of the drying shrinkage has occurred. The steel and concrete stresses are still assumed to follow a parabolic distribution as shown in Figures 4-2b and 4-2c. (Gilbert 1992)

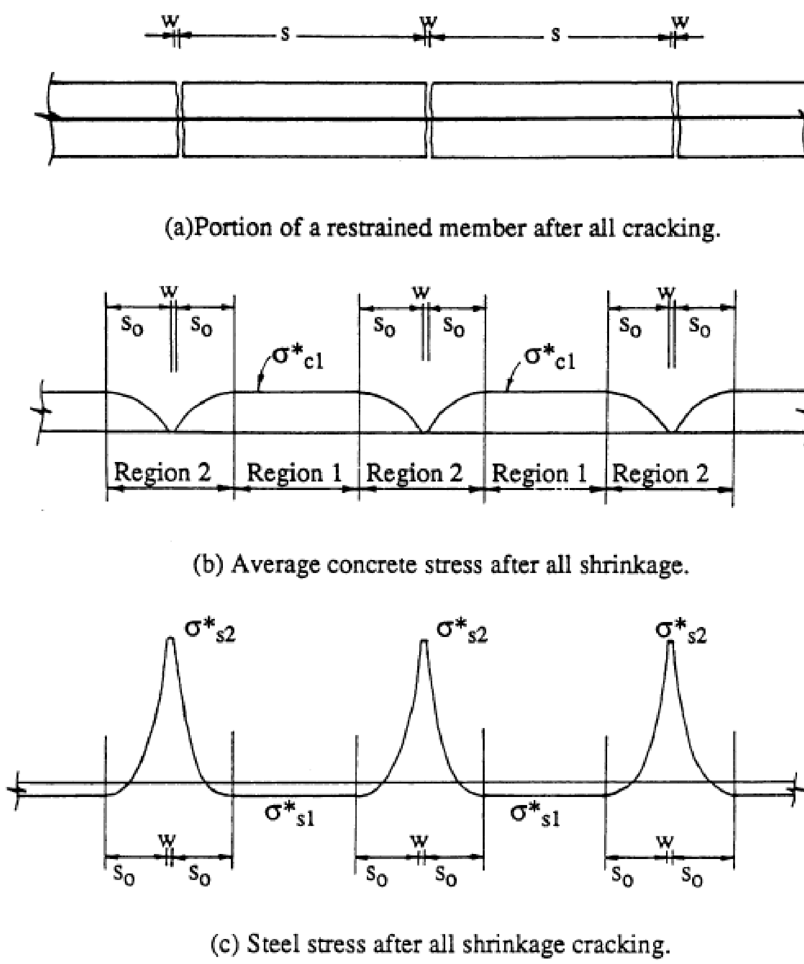


Figure 4-2: Final Cracking Illustration (Gilbert 1992)

4.3 Minimum Reinforcement Ratio Recommendations

Gilbert (1992) examined the following minimum (or temperature and shrinkage) reinforcement recommendations to control direct tension cracking. The equations in Gilbert (1992) were in metric units; however, in this section the author included these equations as well as adding the equations for U.S. customary units. Equation 4-1 is a recommendation from Campbell-Allen and Hughes (1978).

$$\rho_{\min} = \frac{14.45 f_t}{f_y} \qquad \text{(U.S.)}$$

Equation 4-1

$$\rho_{\min} = \frac{1.2 f_t}{f_y} \qquad \text{(metric)}$$

where,

f_t = tensile strength of immature concrete (psi [MPa]) = $0.25\sqrt{f'_c}$),

f'_c = concrete compressive strength (psi [MPa]), and

f_y = reinforcement yield strength (psi [MPa]).

Equation 4-2, 4-3, and 4-4 are recommendations from the Australian code AS 3600 (1988). Equation 4-2 is for fully restrained slabs or slabs where cracks cause aesthetic problems. Equation 4-3 is for slabs in which visible cracks do not cause aesthetic problems. Equation 4-4 is for slabs that are unrestrained and are allowed to freely expand or contract.

$$\rho_{\min} = \frac{360}{f_y} \qquad \text{(U.S.)}$$

Equation 4-2

$$\rho_{\min} = \frac{2.5}{f_y} \qquad \text{(metric)}$$

$$\rho_{\min} = \frac{200}{f_y} \qquad \text{(U.S.)}$$

$$\rho_{\min} = \frac{1.4}{f_y} \qquad \text{(metric)}$$

Equation 4-3

$$\rho_{\min} = \frac{100}{f_y} \qquad \text{(U.S.)}$$

$$\rho_{\min} = \frac{0.7}{f_y} \qquad \text{(metric)}$$

Equation 4-4

Table 4-1 summarizes the minimum reinforcement values from the above equations if AEB Culvert at 240+37 were considered along with AASHTO and ACI recommendations summarized in Sections 2.5.1 and 2.7.4. It should be noted that Section R7.12.1.2 of the commentary of ACI 318 (2011) suggests that the recommendation of 0.0018 and may need to be increased when the member is significantly restrained. The ACI 350R (201) recommendations are done by section lengths between joints as seen in Table 2-1. AEB Culvert at 240+37 used Grade 60 reinforcement (414 MPa), had a concrete compressive strength of 5,200 psi (35.9 MPa), and an immature concrete tensile strength of 18 psi (0.124 MPa).

Table 4-1: Minimum Shrinkage and Temperature Reinforcement Ratios

Minimum Shrinkage and Temperature Reinforcement Ratios	
Source	Reinforcement Ratio
AASHTO (2007) and ACI 318 (2011)	0.0018
ACI 224.1R (2001)	0.0060
ACI 318 (2011)	0.0020 (longitudinal reinforcement in walls)
ACI 350R (2001)	0.0030-0.0050 (environmental structures)
AS 3600 (1988)	0.0060 (fully restrained)
AS 3600 (1988)	0.0033 (no aesthetic problems)
AS 3600 (1988)	0.0017 (unrestrained)
Campbell-Allen and Hughes (1978)	0.0043

4.4 Approach Proposed for Partially Restrained Concrete Subjected to Thermal and Drying Shrinkage

Gilbert (1992) developed a method to estimate the average crack width, crack spacing, concrete stresses, and steel stresses in fully restrained, reinforced concrete members subjected to direct tension. Gilbert assumed that the members are initially fully restrained and that the stresses are only due to an axial force caused by drying shrinkage. However, culverts in the AEB and Corridor X that were surveyed had transverse contraction joints. Since movement will occur at the contraction joint, culverts with contraction joints will be partially restrained instead of fully restrained. Thermal shrinkage effects are also thought to be a driving force behind the distress experienced; therefore, for the scope of this research, Gilbert's equations were modified to include partially restrained conditions and thermal and drying shrinkage.

All credit is given to Gilbert (1992) for the initial assumptions used to derive the equations in Sections 4.4.4 and 4.4.5.

4.4.1 Notation

The notation and units used for all variables in Section 4.3, 4.4, and 4.5 are listed here.

A_c = concrete cross-sectional area (in.^{2})

A_s = reinforcement cross-sectional area (in.^{2})

d_b = reinforcement bar diameter (in.)

E_c = concrete modulus of elasticity (psi)

E_s = steel modulus of elasticity (psi)

E_e^* = final effective modulus of concrete (psi)

F_B = base friction restraint force (lbs)

F_{sh} = total shrinkage force in the wall and roof components just prior to concrete cracking (lbs)

f_B = base friction restraint distributed axial force (lb/in.)

f_c = compressive strength of concrete (psi)

f_t = direct tensile strength of concrete (psi) = $1.7(f_c)^{2/3}$

f_y = reinforcement yield strength (psi)

H = clear height of the culvert (ft)

K = concrete stiffness (lb/in.)

L = length of member or the length between joints (in.)

m = number of cracks

N_{cr} = restraining force immediately after initial cracking (lbs)

$N(\infty)$ = final shrinkage-induced restraining force (lbs)

n = modular ratio = $\frac{E_s}{E_c}$

n^* = final modular ratio = $\frac{E_s}{E_e^*}$

p_o = axially distributed load (lb/in.)

s = average crack spacing (in.)

s_o = distance from crack over which stresses are affected by cracking (in.)

$T_{\max}$ = maximum concrete temperature (°F)

$T_{\min}$ = minimum concrete temperature (°F)

T_{zero} = concrete zero-stress temperature (°F)

W = width of the base slab (in.)

w = average crack width (in.)

α_t = concrete coefficient of thermal expansion (in./in./°F)

ΔT = concrete temperature change (°F)

Δ = contraction joint displacement of a culvert section (in.)

ε_{cr} = initial creep strain (in./in.)

ε_{ds}^* = final drying shrinkage strain (in./in.)

ε_e = elastic concrete strain (in./in.)

ε_{s1}^* = final steel strain (in./in.)

ε_{sh} = initial total shrinkage strain (in./in.)

ε_{sh}^* = final total shrinkage strain (in./in.)

ε_{th}^* = maximum thermal shrinkage strain (in./in.)

ε_1^* = final concrete strain (in./in.)

ρ = reinforcement ratio

σ_{av} = average tensile concrete stress away from the crack location (psi)

σ_{c1} = initial tensile concrete stress away from the crack location (psi)

σ_{c1}^* = final tensile concrete stress away from the crack location (psi)

σ_{s1} = initial compressive steel stress away from the crack location (psi)

σ_{s1}^* = final compressive steel stress away from the crack location (psi)

σ_{s2} = initial tensile steel stress at the crack location (psi)

σ_{s2}^* = final tensile steel stress at the crack location (psi)

τ_f = base friction restraint factor (psi)

ϕ^* = creep coefficient

4.4.2 Thermal Shrinkage

Gilbert did not include thermal deformations in his total shrinkage term. For the purposes of this thesis, thermal shrinkage was included in the total shrinkage strain to better estimate the effects experienced by cast-in-place (CIP) reinforced concrete box culverts.

Using the concrete temperature change from Equation 2-1 of this thesis, the thermal shrinkage strain was calculated using the following equation from Mehta and Monteiro (2006):

$$\varepsilon_{th}^* = \alpha_t \Delta T$$ **Equation 4-5**

The ΔT was calculated using modeled concrete temperatures for computed placement conditions typical for Alabama using ConcreteWorks software. The ConcreteWorks temperature results are shown in Figure B-1 in Appendix B. The following assumptions were made in calculating the maximum and minimum temperatures using ConcreteWorks:

- The culvert was modeled as a 24 in. thick concrete pavement because it closely resembles the base slab of a box culvert,
- The pavement was placed in Birmingham, Alabama,
- The maximum concrete temperature was taken from a placement on August 15 at 8 a.m., and the minimum concrete temperature was taken from a concrete pavement placed on February 13 at noon,
- The pavement was placed on a 12 in. granular subbase, and a clay subgrade,
- The initial ambient temperature was used as the fresh concrete temperature at placement,
- Wood formwork was used,
- The formwork was removed at a concrete age of 72 hours,
- A double coat curing compound was applied 72 hours after placement,
- A light gray cure method color was used, and
- The maximum and minimum temperatures found were from the middle of the pavement.

With a thermal shrinkage term defined, Gilbert's total shrinkage strain term was redefined as follows:, where thermal shrinkage (ε_{th}^{*}) is the new addition:

$$\varepsilon_{sh}^{*} = \varepsilon_{ds}^{*} + \varepsilon_{th}^{*}$$ **Equation 4-6**

4.4.3 Accounting for Movement at Contraction Joints

The objective of calculating contraction joint movements is to obtain an estimate to integrate into the Gilbert (1992) analysis. The estimate is of the total contraction joint movement in a culvert base section. The stiffness method and matrix structural analysis were used to calculate the contraction joint displacement caused by the partially restrained shrinkage of the concrete. The problem was modeled as a single degree of freedom (SDOF) slab on grade (the base slab of the culvert) with a node at each end and restrained at one end. Modeling as a base slab allowed the restraint provided by the subgrade to be taken into account by using a base friction coefficient. The free-body diagram of the slab and the SDOF free-body diagram are illustrated in Figure 4-3. Figure 4-4 illustrates how equivalent forces from the uniformly distributed load are distributed to the nodes.

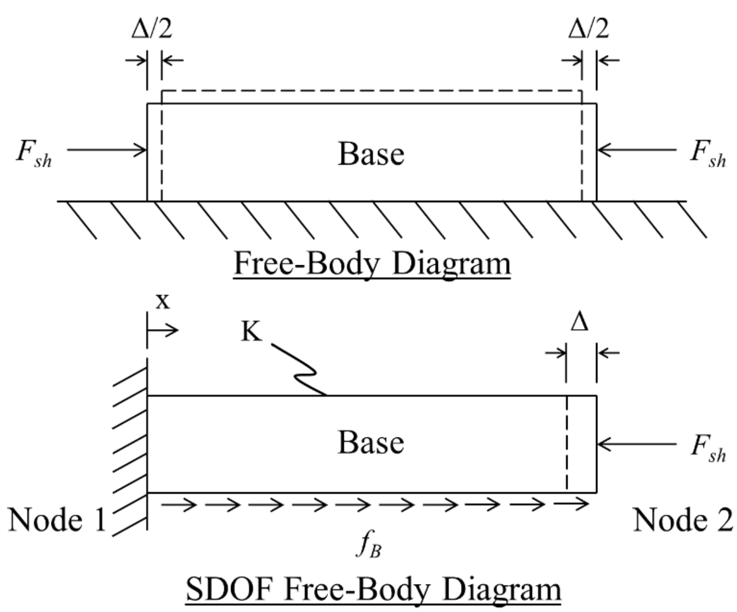


Figure 4-3: Free-Body Diagrams for Displacement Calculations

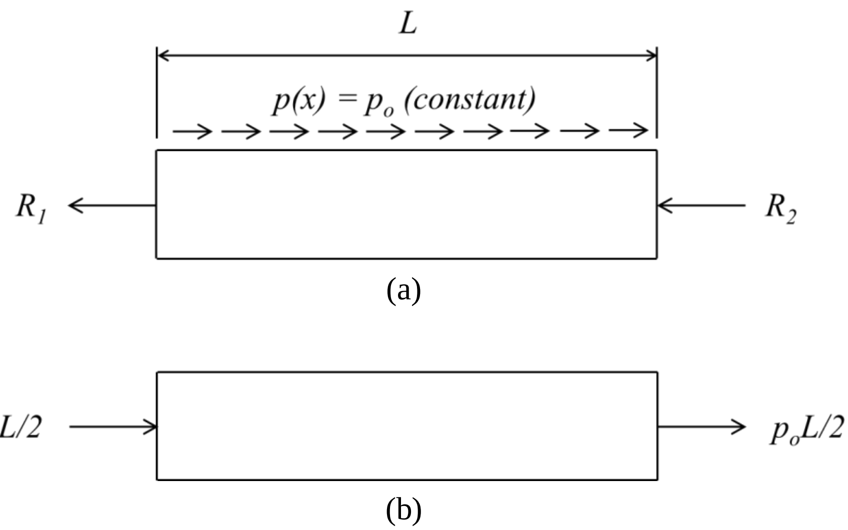


Figure 4-4: Bar Element with a Uniformly Distributed Load: (a) Reaction Forces; (b) Equivalent Forces (Sennett 1994)

In this chapter, the direct tensile strength of the concrete used was estimated using the following equation (Raphael 1984):

$$f_t = 1.7(f_c)^{2/3} \qquad \textbf{Equation 4-7}$$

Using the tensile strength value from Equation 4-7, the concrete shrinkage force was calculated as follows:

$$F_{sh} = f_t A_c \qquad \textbf{Equation 4-8}$$

The shrinkage force F_{sh} estimate is intended to be an upper bound estimate of the force the base needs to carry, which likely corresponds to an upper bound joint movement. The tensile strength f_t of the concrete is used because the stress just before cracking is desired in order to calculate an upper bound value. A_c, as it is assumed that the wall and ceiling placement is cast when the base is mature, is the area of the culvert cross section above the base. The distributed force resisting the shrinkage of the concrete from friction provided by the subbase was calculated as follows:

$$f_B = \tau_f W \qquad \textbf{Equation 4-9}$$

The base friction restraint factor was taken from Figure 4 and Table 3 in Rassmussen and Rozycki (2001). A value of 2.0 psi (14 kPa) was used for a granular subbase. The distributed base friction restraint force needed to be converted to an equivalent concentrated force distributed equally between the two nodes (See Figure 4-4). This was accomplished by using Sennett's (1994) equation as shown in Equation 4-10.

$$F_B = \frac{p_o L}{2} = \frac{f_b L}{2} \qquad \textbf{Equation 4-10}$$

In this chapter, the modulus of elasticity of the concrete was calculated using the ACI 318 (2011) equation for normalweight concrete as shown in Equation 4-11.

$$E_c = 57{,}000\sqrt{f_c} \qquad \textbf{Equation 4-11}$$

The stiffness of the concrete is calculated using Equation 4-12 for a one-dimensional bar element (Sennett 1994).

$$K = \frac{AE}{L}\begin{bmatrix} 1 & 1 \\ -1 & -1 \end{bmatrix} = \frac{A_c E_c}{L}\begin{bmatrix} 1 & 1 \\ -1 & -1 \end{bmatrix} \qquad \textbf{Equation 4-12}$$

The stiffness method proposed by Sennett (1994) was then used to relate the forces, stiffness, and displacements of the base slab as shown in Equations 4-13a and 4-13b. The variables from the base slab problem were substituted into this equation in Equation 4-13c.

$$F = K\Delta \quad \text{(a)}$$

$$\begin{Bmatrix} F_1 \\ F_2 \end{Bmatrix} = \begin{bmatrix} k_{11} & k_{12} \\ k_{21} & k_{22} \end{bmatrix} \begin{Bmatrix} \Delta_1 \\ \Delta_2 \end{Bmatrix} \quad \text{(b)} \qquad \textbf{Equation 4-13}$$

$$\begin{Bmatrix} F_B \\ F_{sh} + F_B \end{Bmatrix} = \frac{A_c E_c}{L} \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \begin{Bmatrix} 0 \\ \Delta \end{Bmatrix} \quad \text{(c)}$$

By applying boundary conditions, which take into account the fixed support at node 1, Equation 4-13c can be simplified to Equation 4-14. Equation 4-14 can then be rearranged to solve for the displacement at node 2, the overall displacement of the slab, in Equation 4-15.

$$(F_{SH} - F_B) = \frac{A_c E_c}{L} \Delta \qquad \textbf{Equation 4-14}$$

$$\Delta = \frac{(F_{SH} - F_B) L}{A_c E_c} \qquad \textbf{Equation 4-15}$$

Some results for movement at contraction joints are presented in Table B-1 in Appendix B.

4.4.4 First cracking

For the crack width and stresses to be calculated, an approximation for the distance, s_o, over which the crack affects the steel and concrete stresses must first be calculated. The following equation used by Gilbert (1992) was taken from Favre et al. (1983):

$$s_o = \frac{d_b}{10\rho} \qquad \textbf{Equation 4-16}$$

Refer to Figures 4-1c, 4-1d, 4-2b, and 4-2c for an illustration of s_o. Stresses that are within s_o from the crack are referred to as being located in Region 1, and stresses that are outside of s_o from the crack are referred to as being located in Region 2 (Gilbert 1992).

Reinforcement in a partially restrained member is allowed to elongate or contract, Δ. Using this concept and Gilbert's assumptions that steel stress/strain is parabolic in Region 2, the following equation can be obtained by integrating the reinforcement strain over the length of the member:

$$\frac{\sigma_{s1}}{E_s}L + \frac{\sigma_{s2}-\sigma_{s1}}{E_s}\left(\frac{2}{3}s_o + w\right) + \Delta = 0$$ **Equation 4-17**

As proposed by Gilbert (1992), neglecting the crack width (w will be much smaller than s_o), Equation 4-17 can be rearranged to get the steel stress in Region 1, as presented in Equation 4-18.

$$\sigma_{s1} = \frac{-2\sigma_{s2}s_o - 3\Delta E_s}{3L - 2s_o}$$ **Equation 4-18**

Gilbert (1992) assumed that the reinforcement carried all of the axial restraining force at the crack after first cracking and developed the following equation:

$$\sigma_{s2} = \frac{N_{cr}}{A_s}$$ **Equation 4-19**

By substituting Equation 4-19 into Equation 4-18, the Region 1 steel stress can be defined as a function of the restraining force at first cracking, as presented in Equation 4-20.

$$\sigma_{s1} = \frac{-2\frac{N_{cr}}{A_s}s_o - 3\Delta E_s}{3L - 2s_o} \qquad \textbf{Equation 4-20}$$

Away from the crack, the sum of the reinforcement and concrete forces must equal the restraining force, as represented in Equation 4-21 (Gilbert 1992).

$$\sigma_{c1}A_c + \sigma_{s1}A_s = N_{cr} \qquad \textbf{Equation 4-21}$$

Using this concept the following equation for concrete stress can be developed by rearranging Equation 4-21 (Gilbert 1992):

$$\sigma_{c1} = \frac{N_{cr} - \sigma_{s1}A_s}{A_c} \qquad \textbf{Equation 4-22}$$

The concept of strain compatibility was used to define the relationship between steel and concrete strain in Region 1 as follows (Gilbert 1992):

$$\frac{\sigma_{s1}}{E_s} = \frac{\sigma_{c1}}{E_c} + \varepsilon_{cr} + \varepsilon_{sh} \qquad \textbf{Equation 4-23}$$

The left side of the equation represents the steel strain and the right side of the equation represents the total concrete strain. The tensile creep and elastic strains are counteracted by the contracting shrinkage strain. Initial cracking occurs when the concrete stress reaches f_t. The elastic concrete strain at this instant can be defined as f_t/E_c. These strain concepts are presented in Equation 4-24. (Gilbert 1992)

$$\varepsilon_{cr} + \varepsilon_{sh} = -\frac{f_t}{E_c} \qquad \textbf{Equation 4-24}$$

Finally, Equations 4-20, 4-22, and 4-24 were substituted into Equation 4-23 and N_{cr} can be solved to produce Equation 4-25.

$$N_{cr} = \frac{[3\Delta E_s(1+\rho n) - 3Lf_t n + 2s_o f_t n]A_c A_s}{2s_o n A_s - 3LnA_s - 2s_o(1+\rho n)A_c} \geq 0$$ **Equation 4-25**

where,

$$\rho = \frac{A_s}{A_c}$$

The constraint that Equation 4-25 must be greater than zero was added to prevent the axial contraction condition causing N_{cr} to go into compression, which is not representative of the behavior sought for this problem (Gilbert 1992).

4.4.5 Final Cracking

A creep coefficient was taken into account for final cracking. It was used to calculate the final effective modulus for concrete as defined in Equation 4-26. (Gilbert 1992)

$$E_e^* = \frac{E_c}{1+\phi^*}$$ **Equation 4-26**

The concrete stresses in Region 1 are continually changing. The stresses increase until they reach the direct tensile strength of the concrete and then suddenly decrease back to σ_{c1} once cracking occurs somewhere in this region. Therefore, the final Region 1 concrete stress after the concrete has finished drying can be taken as the average of σ_{c1} and f_t as shown in Equation 4-27 (Gilbert 1992). This concept is illustrated in Figure 4-5.

$$\sigma_{av} = \frac{\sigma_{c1} + f_t}{2}$$ **Equation 4-27**

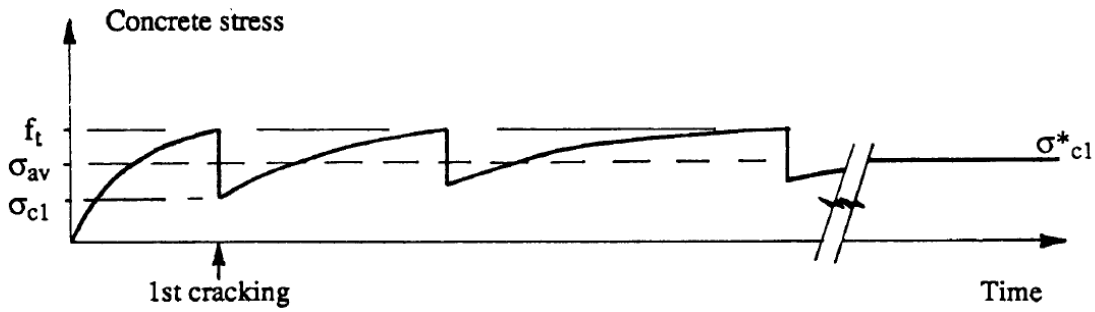


Figure 4-5: Average Concrete Stress (Gilbert 1992)

Knowing the average concrete stress allowed, the number of cracks can be determined by solving the following quadratic:

$$m^2(4s_o{}^2 f_t A_c) + m\lambda - (9L\Delta E_s A_s + 9L^2 E_s A_s \zeta) \geq 0 \qquad \textbf{Equation 4-28}$$

where,

$$\lambda \quad = \quad -6Lf_t A_c s_o + 6\Delta E_s A_s s_o + 6Ls_o E_s A_s \zeta \text{ and}$$

$$\zeta \quad = \quad \frac{\sigma_{av}}{E_e^*} + \varepsilon_{sh}^*$$

It should be noted that s_o in the above equation is the same value that was used in the first cracking equations in Section 4.3.3. The number of cracks from Equation 4-26 can be rounded up to the nearest whole number if desired to obtain the accurate number of cracks in a member. Using Equation 4-28, the crack spacing can now be calculated from Equation 4-29 (Gilbert 1992).

$$s = \frac{L}{m} \qquad \textbf{Equation 4-29}$$

Using the number of cracks from Equation 4-28, the final restraining force $N(\infty)$ and the Region 1 steel stress can be calculated as presented in Equation 4-30 and 4-31.

$$N(\infty) = \frac{-3\Delta E_s A_s - E_s A_s (3L - 2ms_o)\zeta}{2s_o m} \qquad \textbf{Equation 4-30}$$

$$\sigma_{s1}^* = \frac{-2s_o m \sigma_{s2}^* - 3\Delta E_s}{3L - 2s_o m} \qquad \textbf{Equation 4-31}$$

Knowing $N(\infty)$ now allows for the concrete stress and the Region 2 steel stress to be calculated as follows (Gilbert 1992):

$$\sigma_{s2}^* = \frac{N(\infty)}{A_s} \qquad \textbf{Equation 4-32}$$

$$\sigma_{c1}^* = \frac{N(\infty) - \sigma_{s1}^* A_s}{A_c} \qquad \textbf{Equation 4-33}$$

The average crack width can be calculated by integrating the concrete strain over the length of the member as shown in Equation 4-34.

$$w = \frac{-\dfrac{\sigma_{c1}^*}{E_e^*}(L - \dfrac{2}{3}ms_0) - \varepsilon_{sh}^* L - \Delta}{m} \qquad \textbf{Equation 4-34}$$

4.5 Results and Discussion

AEB Culvert at 240+37 (J) was selected for analysis using the equations and approach from Section 4.3. It was chosen because it was the culvert where wide transverse cracking was detected as described in Section 1.1.2. The analysis procedure was used to find the reinforcement percentage that kept the average crack width at the ACI 224 (2001) limit of 0.012 in (0.030 mm) as discussed in Section 1.1.2.

The concrete mixture proportions used for AEB Culvert at 240+37 were used in the ConcreteWorks temperature calculations presented in Section 4.3.2. The exterior wall that is 8 ft (2 m) tall and 15 in. (381 mm) thick, with section lengths of 3 through 7 times the culvert clear height, H, of 8 feet (2 m) was analyzed. The concrete properties, creep coefficient, and final total shrinkage term used in the analysis are summarized in Table 4-2. Grade 60 #4 bars were used for the longitudinal temperature and shrinkage reinforcement. The 28-day strength of 5,200 psi (35.9 Mpa) was obtained as the average from the compressive strength tests performed by ALDOT. The creep coefficient and drying shrinkage quantifications were both calculated using the CEB MC90-99 model (CEB 1999). This model was used because it allowed for a concrete loading age of 1 day to be used for moist-cured concrete when calculating the creep coefficient. The ACI 209R (1992) model only allowed for a loading age of 7 days to be used for moist-cured concrete. Using a loading age of 1 day better characterized the stress relaxation experienced in the concrete due to thermal stresses which occur at these ages. The drying shrinkage strain from the CEB MC90-99 model included the combined effect of drying and autogenous shrinkage. The concrete was exposed to drying at a concrete age of 7 days in the drying shrinkage calculations. An average relative humidity of 70% was used as well. The drying shrinkage strain and creep coefficient are from a concrete age of 365 days. A 28-day compressive strength of 4,000 psi (27.6 MPa) was used for both calculations. The thermal shrinkage assumptions as well as the equation used to calculate the total shrinkage strain are outlined in Section 4.4.2. Drying shrinkage, thermal shrinkage, and creep coefficient calculations are presented in Appendix B.

The contraction joint movement values calculated, the corresponding section lengths, the temperature and shrinkage reinforcement ratio, and the estimated crack width results are presented in Table 4-3. Figure 4-6 summarizes the temperature and shrinkage reinforcement ratio results from the analysis. Example calculations for the contraction joint movement and the temperature and shrinkage analysis are presented in Appendix B. The full contraction joint movement results are tabulated in Table B-1. The full temperature and shrinkage analysis results are tabulated in Table B-2. The sensitivity of the temperature and shrinkage reinforcement to the magnitude of the contraction joint movement is shown in Figure 4-7. The results for Figure 4-7 are included in Table B-3.

Table 4-2: AEB Culvert at 240+37 Concrete Properties, Creep Coefficient, and Final Total Shrinkage

AEB Culvert at 240+37 Concrete Properties, Creep Coefficicent, and Final Total Shrinkage						
f_y (ksi)	f_c (psi)	E_c (psi)	E_s (psi)	d_b (in.)	ϕ^*	ε_{sh}^* (in./in.)
60,000	5,200	4,110,000	29,000,000	0.5	2.11	-594 x 10^{-6}

Table 4-3: Section Lengths, Contraction Joint Movement Values, Reinforcement Ratios, and Estimated Crack Widths for AEB Culvert at 240+37

Section Lengths, Contraction Joint Movement Values, Reinforcement Ratios, and Estimated Crack Widths for AEB Culvert at 240+37				
Spacing	L (ft)	Δ (in.)	ρ	Estimated Crack Width (in.)
3H	24	0.0352	0.0045	0.012
4H	32	0.0466	0.0045	0.012
5H	40	0.0523	0.0048	0.012
6H	48	0.0580	0.0050	0.012
7H	56	0.0692	0.0049	0.012

Figure 4-6: Temperature and Shrinkage Reinforcement Ratio versus Section Length between Joints to Keep the Average Crack Width of AEB Culvert at 240+37 at 0.012 in. (0.30 mm)

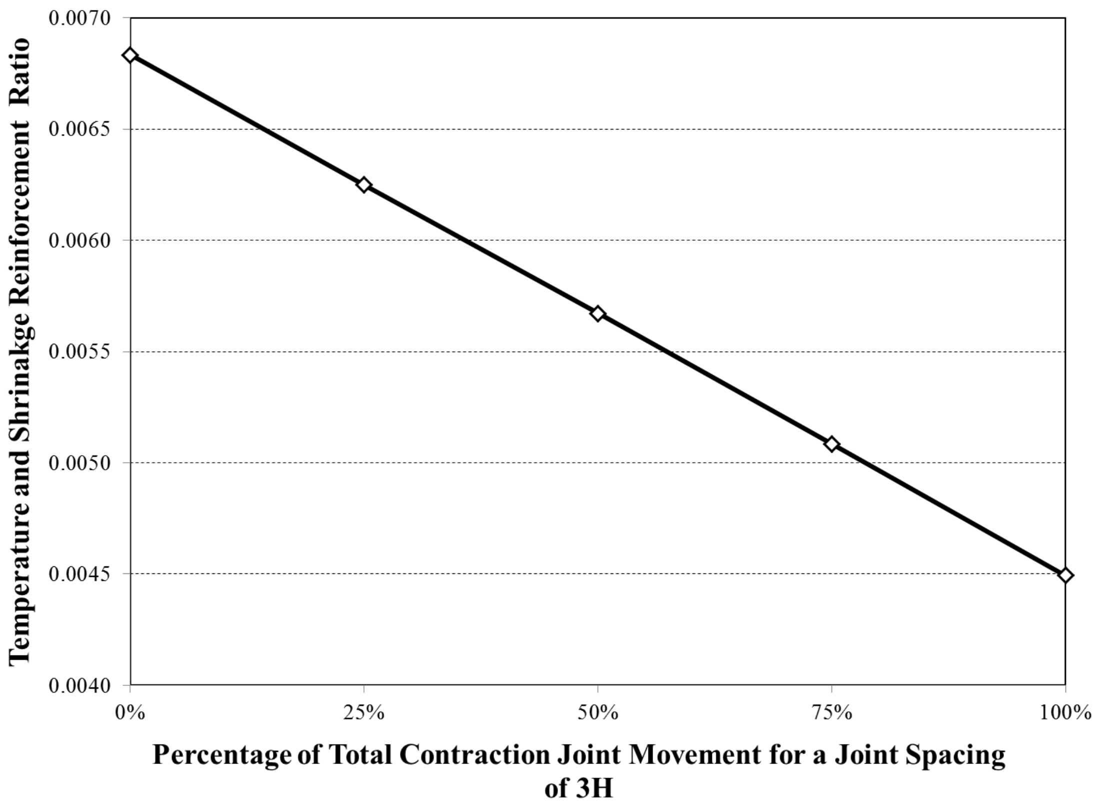


Figure 4-7: Temperature and Shrinkage Reinforcement Ratio to Keep the Average Crack Width at 0.012 in. (0.30 mm) versus Percentage of Contraction Joint Movement for a Joint Spacing of 3H in AEB Culvert at 240+37

The results in Figure 4-6, along with the distress observed in ALDOT CIP reinforced concrete box culverts in Chapter 3, suggest the temperature and shrinkage reinforcement ratio of 0.0021 used in AEB Culvert at 240+37 (other ALDOT reinforced concrete box culverts have been found to have a temperature and shrinkage reinforcement ratio of 0.0018) is not enough to control the widths of cracks that may develop. However, based on the assumed parabolic stress-strain relationship, the estimated steel stress was greater than the yield strength of the reinforcement for each of the section lengths. The σ_{s2}^{*} value from the analysis is not representative of the actual stress that the longitudinal reinforcement in this culvert would experience, but it still raises questions about the reinforcement percentage recommendations. Additional finite element modeling is recommended to quantify the actual steel stresses.

The results in Figure 4-7 indicate that the magnitude of the contraction joint movement has a big effect on the amount of temperature and shrinkage reinforcement needed to keep crack widths at or below 0.012 in (0.30 mm). The analysis was run for a section length of 3 times the clear height of Culvert AEB 240+37, and for contraction joint movements of 0%, 25%, 50%, 75% and 100% of the calculated contraction joint movement (0.0352 in [0.894 mm] for a section length of 3H). The temperature and shrinkage reinforcement ratio required for the 100% contraction joint movement was 0.0045 as compared to 0.0068 for the 0% movement. This was a 34% *decrease* in the temperature and shrinkage reinforcement ratio. There was also a 12% *decrease* when comparing the 100% and 75% values, a 10% *decrease* when comparing the 75% and 50% values, a 9% *decrease* when comparing the 50% and 25% values, and a 9% *decrease* when comparing the 25% and 0% values.

Comparing the temperature and shrinkage reinforcement ratios from Table 4-1 also suggest that the longitudinal temperature and shrinkage reinforcement ratio currently used by ALDOT should be increased even if contraction joints are added. These values also suggest that the reinforcement ratio of 0.0045 recommended in Figure 4-6 for 3 times the clear height of the culvert is reasonable. It is very close to the Campbell-Allen and Hughes (1978) reinforcement ratio of 0.0043, and it is between the range of 0.0030 to 0.0050 recommended by ACI 350R (2001). Their recommendation suggests that the current ALDOT ratio should be increased, too. The AASHTO (2007) and ACI 318 (2011) provision, and the ACI 318 (2011) provision for walls suggest that the current ALDOT ratio is sufficient. However, as stated in Section 2.5.1, Section R7.12.1.2 of the commentary of ACI 318 (2011) suggests that these recommendations may need to be increased when the member is significantly restrained. ACI 224 (2001) recommendations suggest that the current ALDOT value should be increased, and that the ratios in Figure 4-6 are too low as well. The Australian code (AS 3600 1988) provision for unrestrained slabs also suggests that the current ALDOT ratio should be increased because a culvert at least partially restraint. The Australian code AS 3600 (1988) provision for slabs without aesthetic restrictions appears to be applicable to culverts, and it also suggest that the currently-used ALDOT ratio should be increased. The Australian code AS 3600 (1988) provision for fully restrained slabs only applies to culverts that have continuous reinforcement through their transverse joints; however, the values in Figure 4-6 seem reasonable as they are all less than 0.006. This suggests that the ACI 224 (2001) recommendation is too high for culverts built with transverse contraction joints because they are only partially restrained.

The values from Figure 4-6 are conservative, as a worst case temperature difference and drying shrinkage strain are used. From an economic point of view, a more practical temperature and shrinkage reinforcement ratio of 0.0040 should be considered. However, this needs to be verified by additional finite element analysis and subsequent verification based on field-measured behavior.

4.6 Summary and Conclusions

In this chapter, an analysis procedure from Gilbert (1992) for calculating average crack widths in fully restrained concrete members was modified by the author to account for thermal and drying shrinkage and for movement at contraction joints. The modified analysis procedure was used to estimate the temperature and reinforcement percentage required to keep the average crack width of the exterior wall of AEB Culvert at 240+37 at or less than 0.012 in (0.30 mm). The results from the analysis are presented in this chapter. Various temperature and shrinkage reinforcement ratio recommendations were also compared to the current ALDOT ratio of 0.0018 to 0.0021 and to the results from the analysis procedure.

There are three key findings from this chapter. The first is that the temperature and shrinkage reinforcement ratio currently used in CIP reinforced concrete box culverts by ALDOT is not enough to control crack widths to 0.012 in (0.30 mm) or less. The distress observed in the culvert crack condition surveys in Chapter 3, the analysis results, and the various temperature and shrinkage reinforcement recommendations investigated support this conclusion. The second is that the temperature and shrinkage reinforcement ratio results from the analysis are reasonable. The ratio of 0.0045 for a section length of 3 times the clear height of the culverts is consistent with the Campbell-Allen and Hughes (1978) and ACI 350R (2001) recommendations. However, the steel stresses from the analysis were greater than the yield strength of the reinforcement. It is the authors opinion that these stress are not representative of the actual stresses in the temperature and shrinkage reinforcement, but it raised questions about the recommendations from the analysis. The third conclusion is that a temperature and shrinkage reinforcement ratio of 0.0040 for CIP reinforced concrete box culverts should provide adequate crack width control and be economical in culverts with contraction joints at three times the culvert clear height; however, additional finite element analysis and subsequent verification from field-measured behavior is needed to verify this.

Chapter 5

Instrumentation Plan to Determine Field Behavior of Box Culverts

5.1 Introduction

A research objective of this study was to develop an instrumentation plan to assess the development of stresses in newly constructed culverts. To achieve this objective a plan is developed to record the concrete-stress development, strain development, and temperature development for a culvert in the field. The following concrete properties and data will also be obtained using the concrete made at the culvert site: creep, drying shrinkage, tensile-strength development, modulus-of-elasticity development, compressive-strength development, maturity, setting time, total air content, concrete temperature at the time of placement, and the 28-day strength of the concrete. The concrete mixture used at the culvert site will also be reproduced in the lab to document the early-age restrained stress development. This chapter contains the procedures that should be used to perform these tests and gather these data.

Initially it was intended that the author implement this plan for a culvert located in the Birmingham Northern Beltline. However, this project has been delayed, and the data from instrumentation and procedures presented in this chapter will not be included in this thesis. They will be included in future research.

5.2 Instrumentation Plan and Test Procedures

5.2.1 Concrete Stress, Strain, and Temperature

The stress, strain, and temperature development in the concrete of panels (walls) A, B, C, and D of the box culvert will be documented. This is done in order to accurately document the temperature and stress development that causes cracking. The location of these panels is shown in Figure 5-1. Panels A and B will be exterior walls of adjacent culvert barrel sections that are at least 100 ft (30.5 m) from the culvert openings. Panels C and D will be adjacent sections that are placed at a later date than panels A and B and are at least 100 ft (30.5 m) from the culvert openings. The panels will have the maximum restraint possible in these cases, and therefore the worst case stresses. Culvert sections near the entrance will be less restrained than sections deep into the culvert. The restraint in panels at least 100 ft (30.5 m) into the culvert should be as high as it will be anywhere.

The stress, strain, and temperature development data will be collected using 60 vibrating-wire strain (VWS) gauges, each with a 60 ft (18 m) long cable. Fifteen VWS gauges will be used in each of the four panels. The locations of the VWS gauges are illustrated in Figure 5-1. Each VWS gauge is to be attached to reinforcement using zip ties as shown in Figure 5-2. The VWS gauges are spaced horizontally at one-fourth the clear height of the culvert because there is more movement in walls (panels) closer to the contraction joint. This is because there is less restraint at the contraction joint due to joint movement. The culvert wall will also be free to move more at the top of the wall than it will at the bottom, because there is more restraint at the bottom of the wall due to the stiff base; therefore, rows of gauges were put 12 in. (305 mm) from the top and bottom on the culvert wall. The horizontal and vertical spacing of the VWS gauges allow for stress, strain, and temperature development to be documented at various degrees of restraint. Cracks generally begin bottom or top of the wall; therefore, only one VWS gauge was put in the middle of the wall. Two data collection units will be used to collect the data from the VWS Gauges. A data collection unit can accommodate up to 32 VWS Gauges; therefore, one data collection unit will be used at panels A and B, and one at panels C and D. Measurements will be taken in panels A and B at 15-minute intervals for 14 days after the placement of the top slab and walls of panel B. Measurements will be taken in Panels C and D at 15-minute intervals for 14 days after the placement of the top slab and walls of panel D.

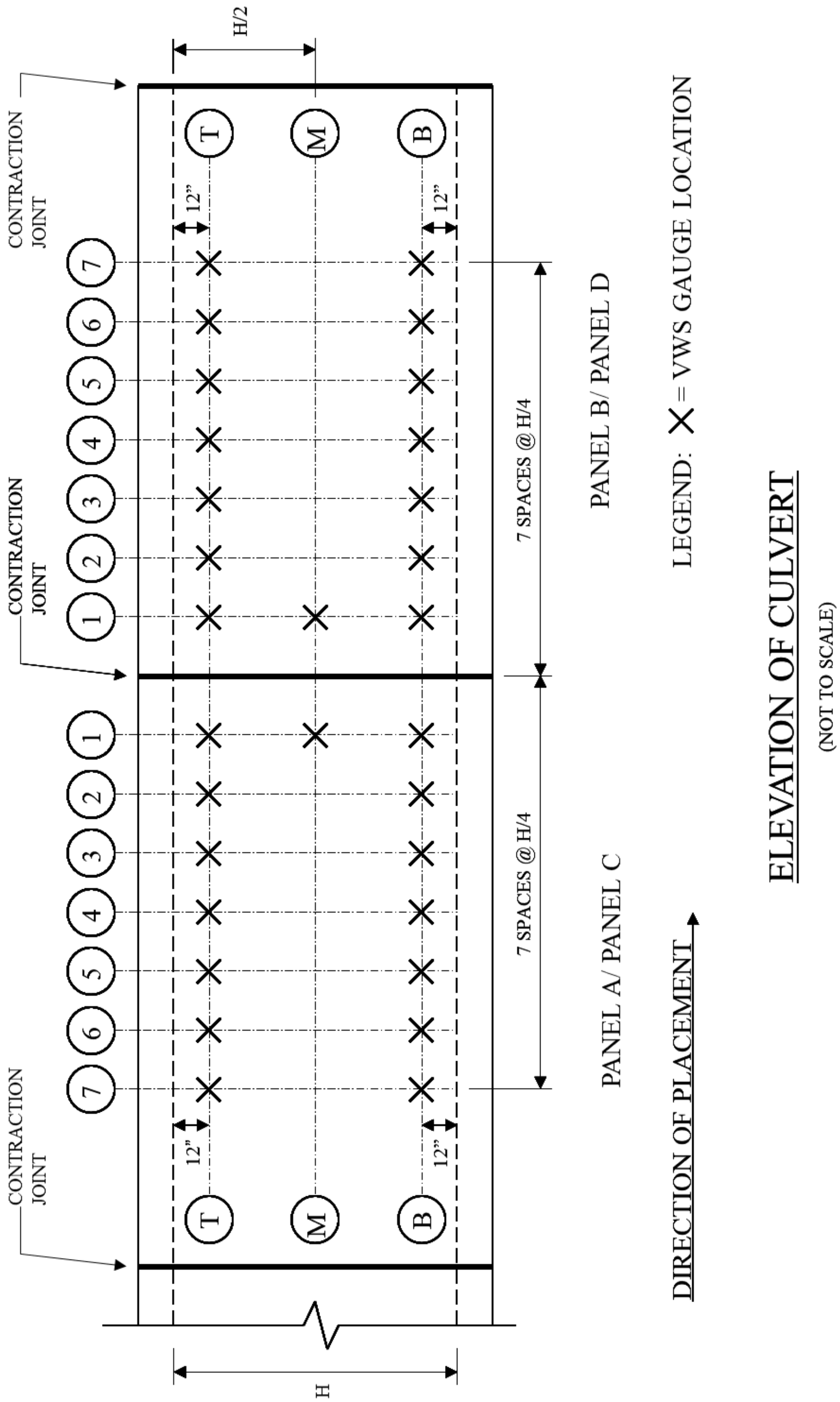


Figure 5-1: VWS Gauge Wall Locations

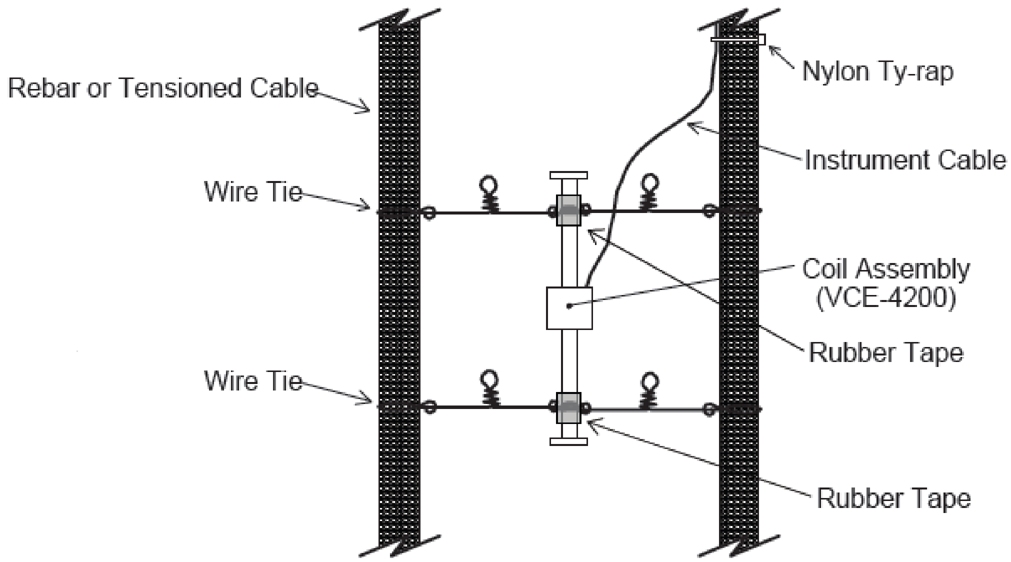


Figure 5-2: VWS Gauge Attachment Illustration (Geokon 2010)

5.2.2 Crack Detection

The occurrence of cracks will be monitored by visual inspection and by evaluating the strain data from the VWS gauges. Each of the panels will be checked for cracks early in the morning and in the afternoon. Visual surveys will also be done throughout the day when the team is at the culvert site.

5.2.3 Crack Width Development

One-inch (254 mm) long 3/8 in. (9.5 mm) diameter stainless steel threaded studs and a demountable mechanical (DEMEC) strain gauge, shown in Figure 5-3, will be used to monitor the progression of crack widths and joint movement in Panels A, B, C, and D of the culvert. The studs will be made from a stainless steel threaded rod cut into 1 in. (25 mm) long sections. A grinding wheel will be used to remove sharp edges from the ends of the cut sections and to make the end sections as flat as possible. A centered hole approximately 3/16 in. (4.8 mm) deep will be drilled in the top of each stud with a 1/32 in. (0.79 mm) drill bit in order to form an indention that enables readings to be taken with a DEMEC gauge. Using cutting oil when drilling these holes provides better drilling performance. Two DEMEC studs are shown in Figure 5-4.

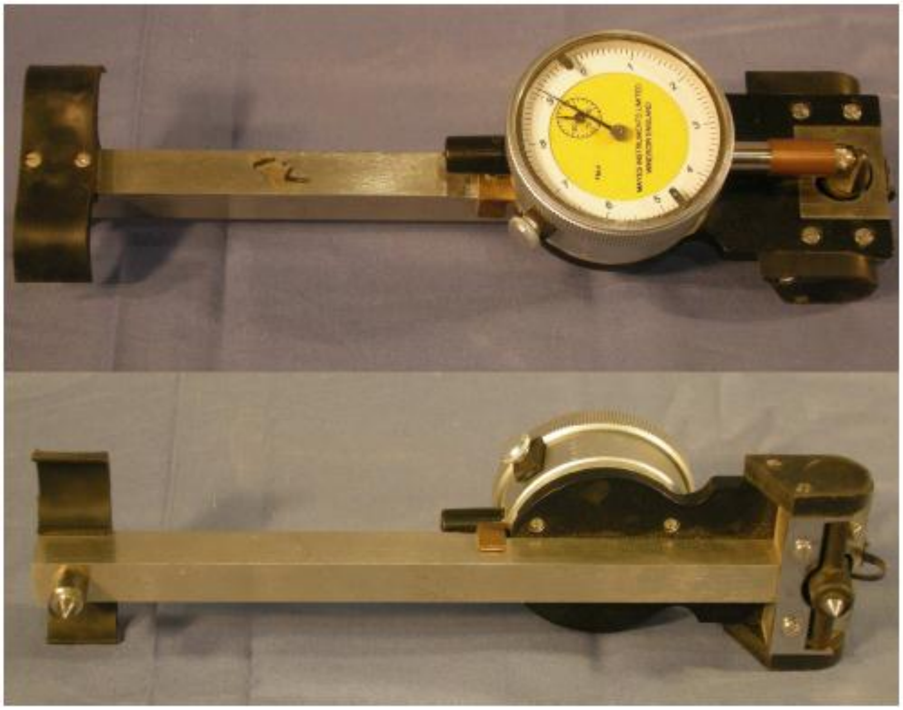


Figure 5-3: DEMEC Gauge (Kavanaugh 2008)

Figure 5-4: Stainless Steel DEMEC Stud

At the culvert site, the stainless steel studs will be epoxied into drilled holes in the culvert panels as shown in Figure 5-5. The holes will be drilled 7/8 in. (22 mm) deep into the concrete with a hammer drill using a 3/8 in. (9.5 mm) drill bit. Once the epoxy has dried, a DEMEC gauge will used to take the crack width measurements. The studs will be installed as shown in Figure 5-6. Studs in Figure 5-6 with a crack or joint between them will be used to measure the width of the crack or joint opening over time. Studs in Figure 5-6 without a crack or joint between them will be used to measure any concrete volumetric change (thermal effects, etc.) so that it can be used to account for temperature effects when determining the crack and joint width. There will be two volumetric change studs at joints, one on each side of the joint, because the concrete on each side will come from two different concrete pours. Any movement found from the volumetric change studs will be subtracted from the measurement taken from the studs around the crack or joint.

Figure 5-5: DEMEC Stud Installed in Concrete

Crack width measurements will be taken at the same intervals outlined for drying shrinkage prisms in ASTM C 157 (2006). The difference is that the measurements will start on the day that the crack is detected.

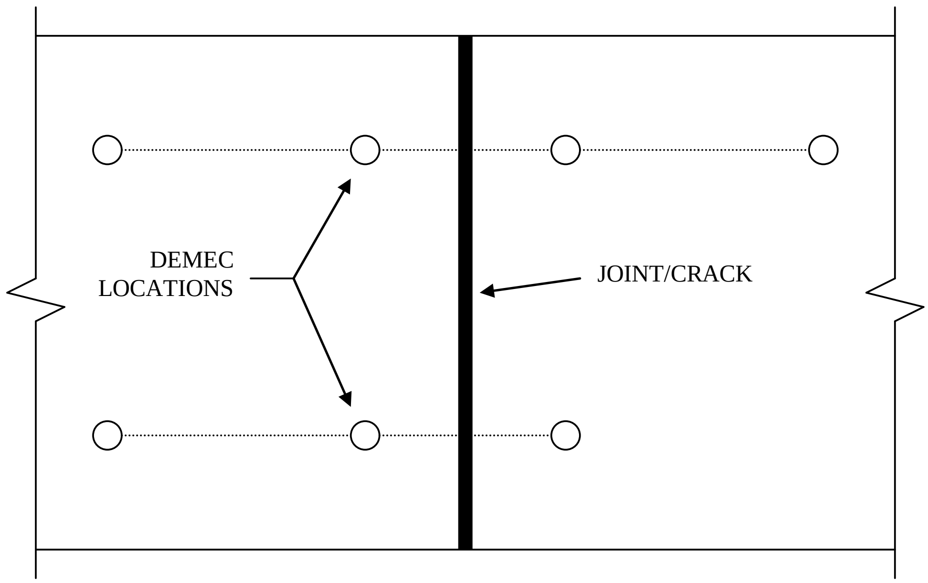


Figure 5-6: DEMEC Stud Wall Layout (Top Row – Construction Joint Configuration; Bottom Row – Crack Configuration)

5.2.4 Creep Testing

Creep testing will be performed on concrete from panels B and D. The procedure described in ASTM C 512 (2002) will be followed, with the exception that the creep specimens will be loaded at concrete ages of 3 and 7 days. Four creep frames will be used in the creep testing. An illustration of a creep frame is shown in Figure 5-7.

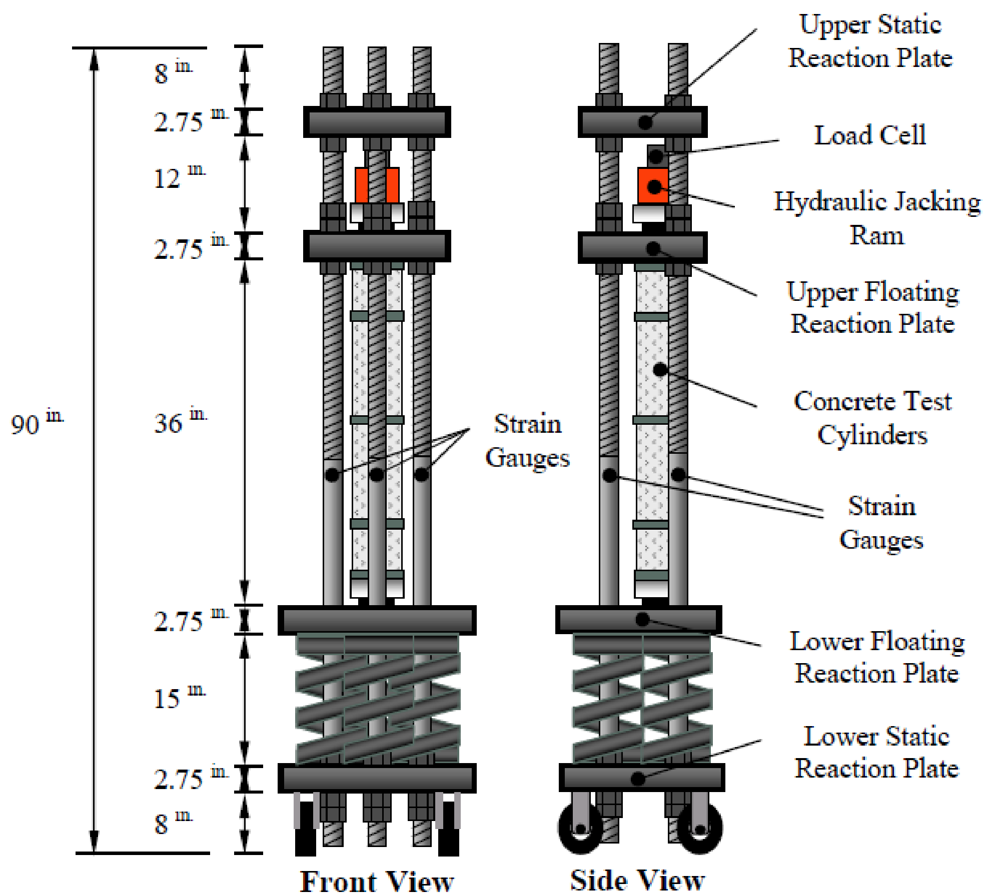


Figure 5-7: Elevation View of a Creep Frame (Kavanaugh 2008)

Seven 6 in. x 12 in. (152 mm x 305 mm) cylinders will be made for each creep frame (28 total cylinders) at the construction site. These cylinders will be transported from the culvert site to Auburn University. The cylinders will be covered and stored in an insulated container to prevent evaporation during transportation. After they are demolded, the cylinders will be moist cured until it is time to prepare them for creep testing. Two of the seven cylinders will be used to determine the compressive strength of the concrete, two will be loaded into the creep frame, and three will be used for drying shrinkage measurements. Both ends of the compressive strength and creep frame cylinders will be put into a cylinder-grinding machine to make the ends flat and perpendicular to the axis of the cylinder. DEMEC points will be epoxied onto the surface of the drying shrinkage and creep cylinders at 120^{o} intervals. Two DEMEC points, spaced 8 in. (203 mm) apart, will be placed at each interval section. This is illustrated in Figure 5-8.

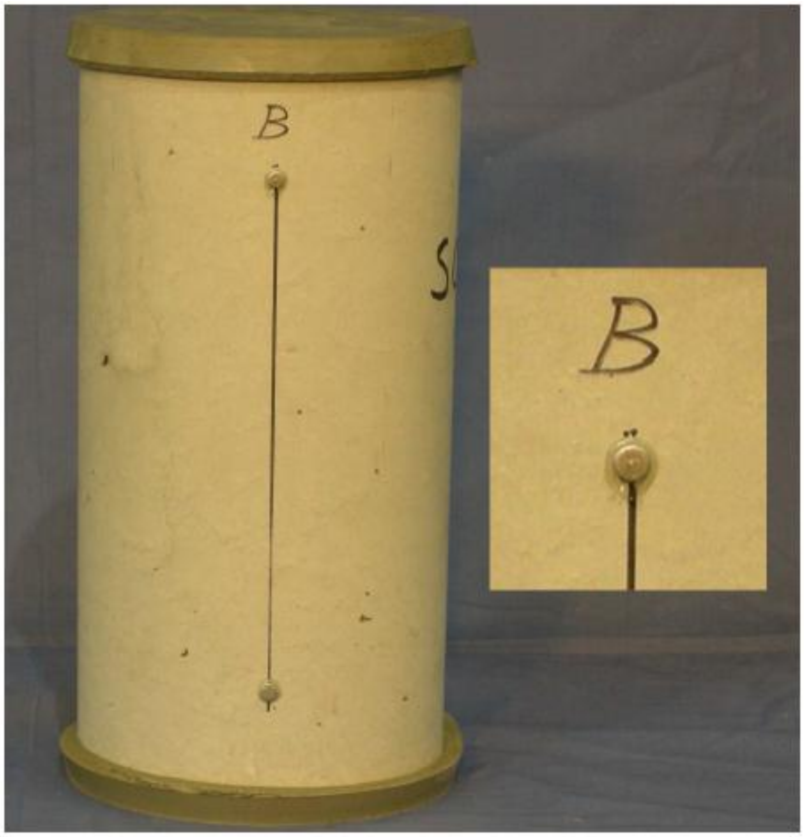


Figure 5-8: A Concrete Cylinder with DEMEC Points (Kavanaugh 2008)

The concrete creep specimens will be loaded with a hydraulic jacking ram to 102% of the target creep frame load, which ASTM C 512 (2002) specifies as 40% of the average strength of the two compressive strength cylinders. The applied load will be measured by a load cell that is placed on top of the hydraulic jacking ram during loading (refer to Figure 5-7 for an illustration). Loading the creep specimens to 102% of the target load takes into account the slight load reduction that occurs when the hydraulic jacking ram is removed. When the target load is achieved, the nuts on the Upper Floating Reaction Plate (see Figure 5-7) will be hand tightened, and the hydraulic jacking ram will be removed. Strain gauges attached to the steel rods on the creep frame (see Figure 5-7) will be used to monitor the load in the creep frame after the hydraulic jacking ram has been removed. Initial strain gauge readings will also be taken before the cylinders are loaded. The strain gauge readings will be used to determine whether or not the load in the creep frame is within 2% of the target load, the tolerance specified in ASTM C 512 (2002). If it is not, the frame will be reloaded.

5.2.5 Drying Shrinkage

Drying shrinkage data from concrete prisms will be obtained for specimens from panels B and D. The procedure detailed in ASTM C 157 (2006) will be followed, with the following exceptions:

- The prisms will be exposed to drying at concrete ages of 3 and 7 days.
- The prisms will be cured in a lime bath from the time they are demolded until testing time.
- The initial drying shrinkage reading will be taken when a prism is first exposed to drying, or taken out of the lime bath.

The prisms will be exposed to drying at 3 and 7 days in order to have more accurate drying shrinkage data that correspond with the creep testing. A total of 12 drying shrinkage prisms will be made at the culvert site (three 3-day panel B prisms, three 7-day panel B prisms, three 3-day panel D prisms, and three 7-day panel D prisms). These prisms will be transported to Auburn University while in molds and wrapped in damp burlap cloth. The prisms will be removed from the molds upon arrival.

5.2.6 Tensile Strength, Modulus of Elasticity, and Compressive Strength Development

For this study, 6 in. x 12 in. (152 mm x 305 mm) concrete cylinders from panels B and D will be tested at the ages of 1, 3, 7, 14 and 28 days to determine the modulus of elasticity, splitting tensile, and compressive strength development of the concrete used in the culvert. The average results from two cylinders will be used for each testing age. The cylinders will be made at the culvert site and transported to Auburn University at the ages of 1 and 3 days for testing. They will be cured outside to best resemble in-place conditions. All of the cylinders will be demolded when the forms are stripped from the culvert, except when testing requires that the molds be removed earlier. Twenty cylinders each from panels B and D (40 cylinders total) will be required to perform the tests.

The splitting tensile strength tests will be run first. The procedure in ASTM C 496 (2004) will be followed for splitting tensile strength testing. An aligning jig, shown in Figure 5-9, will be used in aligning the test setup and in loading. A diametric load will be applied with a Forney FX600 compressive testing machine, shown in Figure 5-10 below, along the length of the concrete cylinder until the cylinder fails.

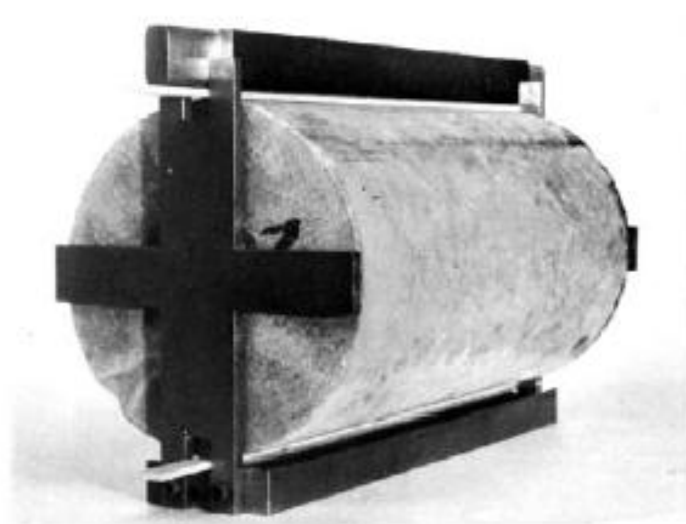


Figure 5-9: A Concrete Cylinder in an Aligning Jig (ASTM C 496 2004)

Figure 5-10: Forney FX600 Compressive Testing Machine

Next, the modulus of elasticity tests will be performed. The procedure from ASTM C 469 (2002) will be followed for modulus of elasticity testing. A Forney FX600 compressive testing machine will be used to load the cylinder and a compressometer, as shown in Figure 5-11, will be used for taking measurements. Each cylinder will be loaded to 40% of the compressive strength, the value specified in ASTM C 469 (2002), for its corresponding age. However, with the compressive strength not known, it will be estimated for the first cylinder from the completed splitting tensile strength. The estimated compressive strength is obtained by rearranging Equation 5-1 (ACI 207 1995) into Equation 5-2.

$$f_{ct} = 6.7\sqrt{f_c}$$ **Equation 5-1**

$$f_c = \left(\frac{f_{ct}}{6.7}\right)^2$$ **Equation 5-2**

where,

f_{ct} = concrete splitting tensile strength (psi) and

f_c = concrete compressive strength (psi).

Four total loadings will be performed on each cylinder, with the first loading being a test loading to check the compressometer gauges. The three real loadings will consist of the cylinder being loaded to 40% of the concrete compressive strength and then being unloaded. During these loadings stress and longitudinal strain that corresponds to 40% of the concrete compressive strength, and the stress that corresponds to a longitudinal strain reading of 50 millionths are to be recorded. These values will be used to calculate the modulus of elasticity as defined in Equation 5-3. (ASTM C 469 2002)

$$E_c = \frac{S_2 - S_1}{\varepsilon_2 - 0.00005}$$ **Equation 5-3**

where,

E_c = concrete modulus of elasticity (psi)

S_1 = stress corresponding to 40% of the concrete compressive strength (psi),

S_2 = stress corresponding to a longitudinal strain of 50 millionths (psi), and

ε_2 = longitudinal strain corresponding to S_2.

The average of the six real readings for the two cylinders will be taken as the modulus of elasticity for the panel.

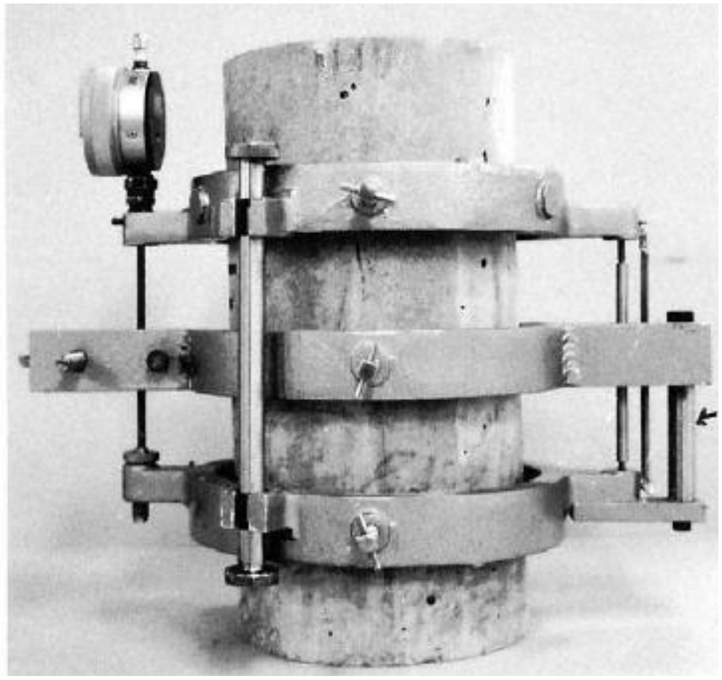


Figure 5-11: Concrete Cylinder in a Compressometer (ASTM C 469 2002)

Finally, the compressive strength tests will be done. The procedure detailed in ASTM C39 (2009) will be followed for compressive strength testing. The cylinders used in the modulus of elasticity tests will be used for the compressive strength tests. A Forney FX600 compressive testing machine will also be used in compressive strength testing to load the cylinders until failure. Before loading a cylinder into the testing machine, it will be placed on a bearing plate and powder will be applied to both ends of the cylinder to reduce friction. The cylinder will then be centered in the test machine and loaded. The target load rate is approximately 1000 lb/s (454 kg/s) for the 6 in. x 12 in. (152 mm x 305 mm) cylinders (ASTM C39 2009).

5.2.7 Maturity

Maturity testing will be done to account for time and temperature effects on the concrete properties in the culvert. Thermocouples will be placed in two concrete cylinders, one cylinder from panel B and one from panel D, made at the culvert site to measure and record the concrete temperature over time. The temperature data obtained will then be related to concrete strength results obtained from the compressive strength testing (see section 5.2.6). These data will be used to develop a strength-maturity relationship that can be applied to the in-place concrete. The temperature development of the in-place concrete will be measured using data from the VWS gauges, and the in-place strength of the concrete will then be estimated using the strength-maturity relationship.

5.2.8 Concrete Setting Times

The time that the concrete takes to set in the field will be documented. The procedure detailed in ASTM C 403 (2006) will be followed for this procedure. One mortar sample will be prepared in the field for panels B and D.

5.2.9 Quality Control Testing

Quality Control testing will be performed by ALDOT in order to obtain the total air content, the fresh temperature at placement, and the 28-day strength of the concrete.

5.2.10 Early-Age Restrained Stress Development

Raw materials will be used to produce the same concrete mixture in the Auburn University structures lab that will be used at the culvert site for panels B and D. The concrete will be placed in two of Auburn University's rigid cracking frames (one for each panel) to monitor and document the development of early-age restrained stress. See Figure 5-12 for a rigid cracking frame. The temperature data in the field from the time of placement of the walls and top slab of panel B to 14 days after that point will be recorded. The same will be done for panel D. The temperatures recorded will then be reproduced in the rigid cracking frames for each panel to assess the early-age behavior of the concrete.

Figure 5-12: Rigid Cracking Frame at Auburn University

Chapter 6

Recommendations to Mitigate Culvert Cracking

6.1 Introduction

One of the research objectives was to develop methods to mitigate cracking in ALDOT cast-in-place (CIP) reinforced concrete box culverts. This chapter presents practices to meet this objective.

6.2 Recommendations

After consulting the practices of AASHTO and of SASHTO states, it was determined that the jointing practices used in ALDOT CIP reinforced concrete box culverts in Alabama should be improved. The data from the culvert crack condition surveys in Chapter 3 indicated that transverse contraction joints should be included in the culvert barrel to alleviate distress. The culvert crack condition surveys also indicated that distress was experienced in the culvert wingwalls. The wingwall jointing practices of SASHTO states indicated that adding expansion and contraction joints may be a way to mitigate wingwall cracking.

6.2.1 Use of Transverse Contraction Joints

As stated in Section 2.5.2.2, contraction joints are used to control cracking and alleviate the stresses due to thermal and drying shrinkage volume change effects (ACI 224 1995). The data gathered from the box culvert condition surveys in Chapter 3 supports that using contraction joints controls cracking. Using contraction joints reduced crack widths in the AEB project by

48% in the *walls* and 43% in the *ceiling* when compared to culverts that used vee joints (or construction joints with continuous reinforcement). Using contraction joints also reduced crack widths in the Corridor X project by 54% in the *walls* and 36% in the *ceiling*. This information suggested that using contraction joints would achieve the objective in stated in Section 6.1, and led to a contraction joint being developed. The proposed transverse contraction joint is designed to allow the culvert sections to contract, have a smaller joint spacing than what is currently used by ALDOT, be sealed so as to prevent outside material from entering the culvert, and to allow adequate load transfer.

6.2.1.1 Transverse Contraction Joint Spacing

After investigating numerous contraction joint spacing recommendations, it was concluded that the 40 to 55 ft (12 to 17 m) transverse joint spacing used in ALDOT Standard Drawing CS-3-1 (2010) was too large; therefore the maximum transverse contraction joint spacing in is recommended as defined in Equation 6-1.

$$s_{max} = \begin{cases} 24 \text{ ft} & \text{for } H \leq 8 \text{ ft} \\ 3H & \text{for } 8 \text{ ft} < H < 12 \text{ ft} \\ 36 \text{ ft} & \text{for } H \geq 12 \text{ ft} \end{cases}$$ **Equation 6-1**

where,

s_{max} = Maximum contraction joint spacing (ft) and

H = Clear barrel height of the box culvert (ft).

Figure 6-1 illustrates the clear height of a culvert and the transverse contraction joint spacing. Three times the clear height of the culvert (or wall height) is the *maximum* contraction joint spacing recommended by ACI 224 (2001). The lower bound constraint of 24 ft (7.3 m) is introduced as ALDOT uses culverts with clear barrel heights less than 8 ft (2 m) and a contraction joint spacing of less than 24 ft (7.3 m) is deemed uneconomical. The upper bound constraint of 36 ft (11 m) is between the 30 to 40 ft (9.1 to 12 m) spacing recommended by the TDOT (2006), and it is divisible by 3.

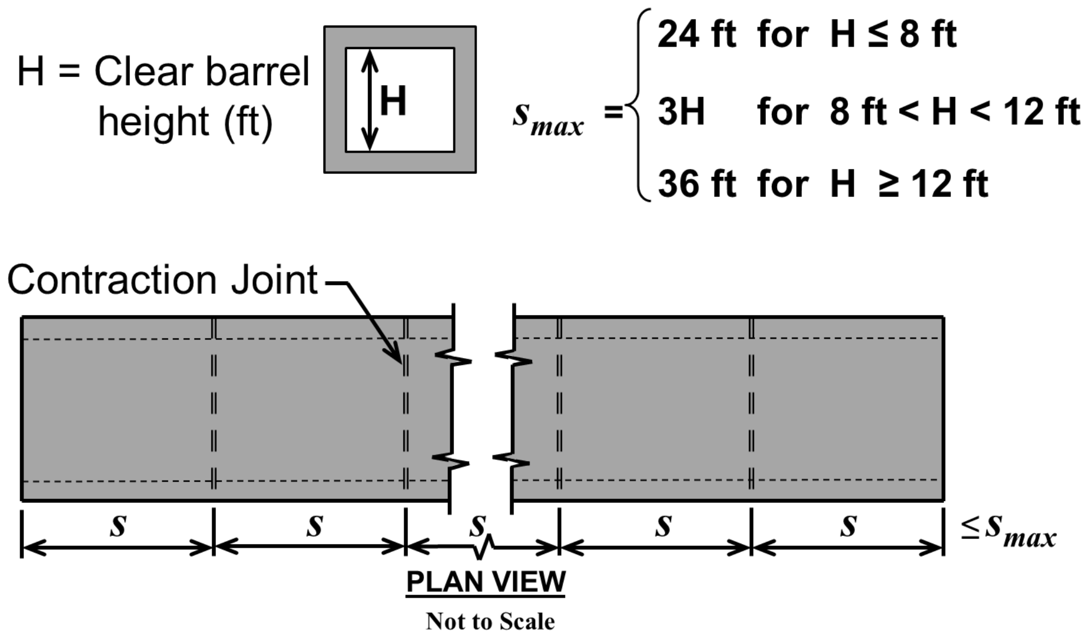


Figure 6-1: Recommended Contraction Joint Spacing and Box Culvert Clear Height

Table 6-1: Transverse Joint Spacing for SASHTO States

Transverse Joint Spacing of SASHTO States		
State	**Transverse Joint Spacing**	**Joint Type**
Alabama (ALDOT 2010)	40-55 ft	Vee
Florida (FDOT 2010c)	30 ft minimum	Vee
Georgia (GDOT 2010)	30 ft maximum	Construction*
North Carolina (NCDOT 2007)	70 ft maximum	N/A
Tennessee (TDOT 2006)	30-40 ft	Contraction
Virginia (VDOT 2008)	25 ft maximum	N/A

* The GDOT spacing applies to construction joints with and without continuous reinforcement

6.2.1.2 Transverse Contraction Joint Design

The transverse contraction joint, shown in Figure 6-2, is proposed for use in the walls, base, and ceiling of ALDOT CIP reinforced concrete box culverts.

No reinforcement should be continuous through this joint. This is to reduce restraint when volume changes occur, and it is consistent with what is specified by the TDOT (2006). A key shall also be provided at the joint to transfer loads across the joint (ACI 224 1995). The width and depth of the key are consistent with the joint used in the plans for AEB Culvert 257+69 (ALDOT 2001), and in Corridor X Culverts 4877+13 (ALDOT 2000a) and 4959+43 (ALDOT 2000b), as shown in Figures 3-15, 3-23, and 3-28. The key shall be beveled to make the joint fit together more easily and minimize the risk of locking-up the joint. The ½ in. bevel dimension was the same as used in the La DOTD joints in Figure 2-37 of this thesis (La DOTD 2008). The ½ in. wide and 1 in. deep groove is consistent with the plans for AEB Culvert 257+69 (ALDOT 2001) and in Corridor X Culverts 4877+13 (ALDOT 2000a) and 4959+43 (ALDOT 2000b).

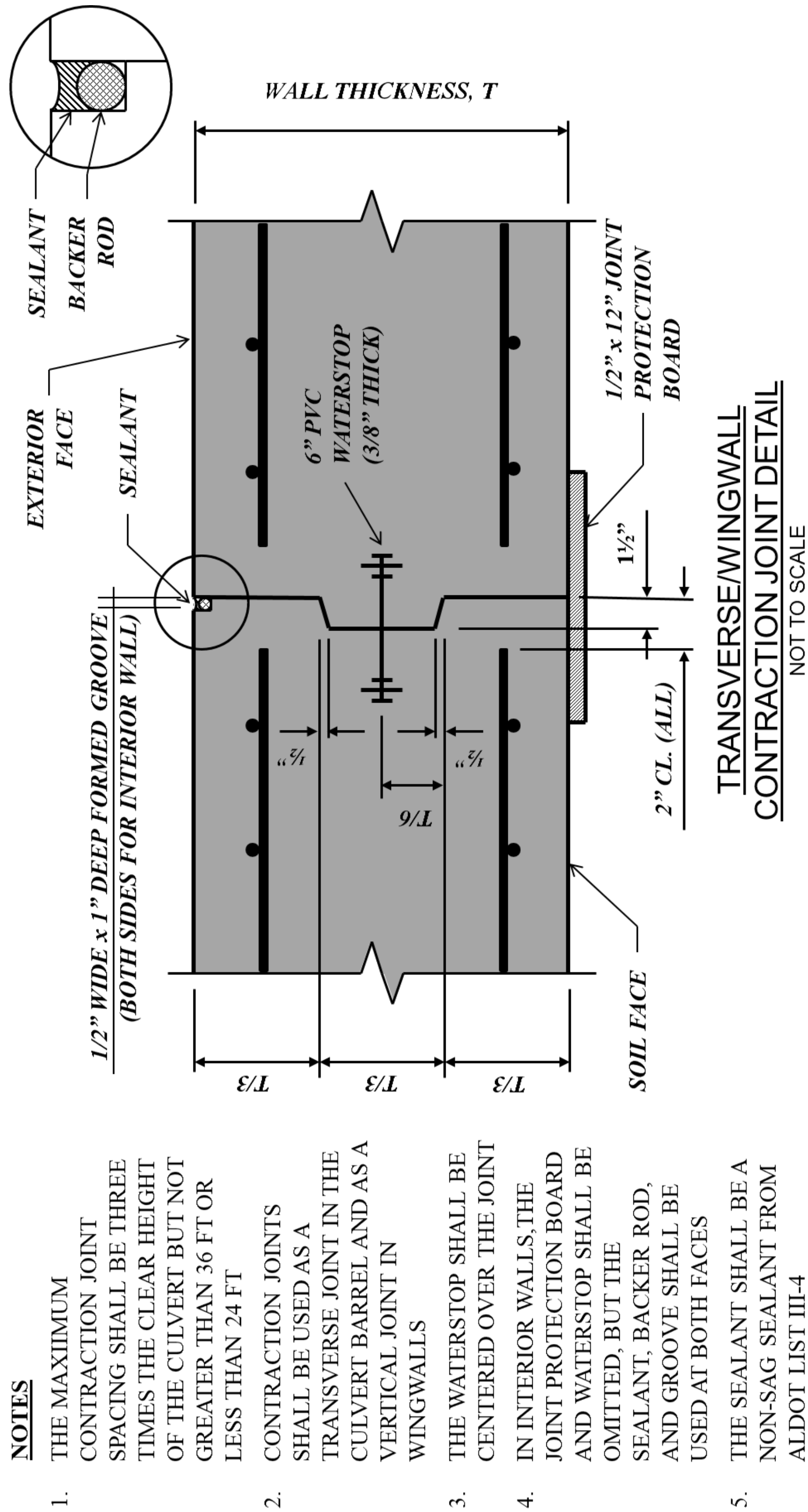


Figure 6-2: Transverse/Wingwall Contraction Joint (Not to Scale)

The contraction joint should be sealed to prevent outside materials from entering through or collecting in the joints of the culvert (ACI 504 1990). The contraction joint should also be sealed to prevent water from eroding the supporting and surrounding soil, which could lead to settlement issues. As recommended in the plans for Culverts AEB 257+69 (ALDOT 2001), Corridor X 4877+13 (ALDOT 2000a), and Corridor X 4959+43 (ALDOT 2000b), the sealant should be a non-sag sealant that is on ALDOT List III-4 (2012b). A non-sag sealant should be used to prevent the sealant from running out of the joint. The sealant should also be able to accommodate the expected joint movements (ACI 504 1990). The sealant should be used at both faces of the joint in interior walls because both faces will be exposed to water. A backer rod should be provided for the sealant, as shown in Figure 6-2, to hold the sealant's shape and to prevent the sealant from bonding with the bottom of the joint (ACI 504 1990). A 6 x 3/8 in. PVC or rubber waterstop shall also be provided. The waterstop is used to prevent water from leaking through the joint (ACI 504 1990). The dimensions of the waterstop and the PVC option are consistent with the plans of Culverts AEB 257+69 (ALDOT 2001), Corridor X 4877+13 (ALDOT 2000a), and Corridor X 4959+43 (ALDOT 2000b). The TDOT allowed for PVC or rubber water stops to be used in construction or expansion joints (see Figure 2-48 and 2-49 of this thesis) (TDOT 2000d). According to ACI 504 (1990), PVC is the most widely used waterstop material (because it can easily be spliced at the work site and special configurations can be made for joint intersections), but rubber has the benefit of being more elastic than PVC. Both PVC and rubber are flexible waterstop materials, and either may be used as long as they form a continuous waterproof seal and allow the joint to move without being damaged (ACI 504 1990; AASHTO 2010a).

Improper installation of the joint sealant and backer rod was noticed in the Corridor X Culvert at 4877+13 as shown in Figure 3-24. Inspection during construction should be used to ensure the correct installation of the backer rod and joint sealant. Improper joint sealing could cause long-term maintenance issues

A ½ in. by 12 in. joint protection board, made of asphalt-impregnated board conforming to ASTM D994, should be used at joint faces that are exposed to soil. This is consistent with what is specified in the plans for Culverts AEB 257+69 (ALDOT 2001), Corridor X 4877+13 (ALDOT 2000a), and Corridor X 4959+43 (ALDOT 2000b). The joint protection is used to keep debris from entering the joint during periods when the joint is open.

6.2.2 Wingwall Joints

Due to the distress observed in wingwalls in the box culvert condition surveys, vertical joints should be used in CIP reinforced concrete box culvert wingwalls for cracking mitigation purposes. These joints are intended to alleviate the stresses from restrained concrete volume change effects and from foundation settlement that may occur.

Joints at the intersection of the culvert and wingwall, and in the wingwall itself, are proposed. Due to the uncertainty in the cause of the wingwall distress, two proposals are made for the vertical joint at the culvert/wingwall intersection. One option is an expansion joint where the wingwall is designed as a free-standing wall. The other is a wing tab joint that was designed by ALDOT. Both of these options will allow for the culvert to rotate at the culvert/wingwall intersection if settlement occurs. They also allow for concrete volumetric changes to occur. The location of the joints is shown in Figure 6-3.

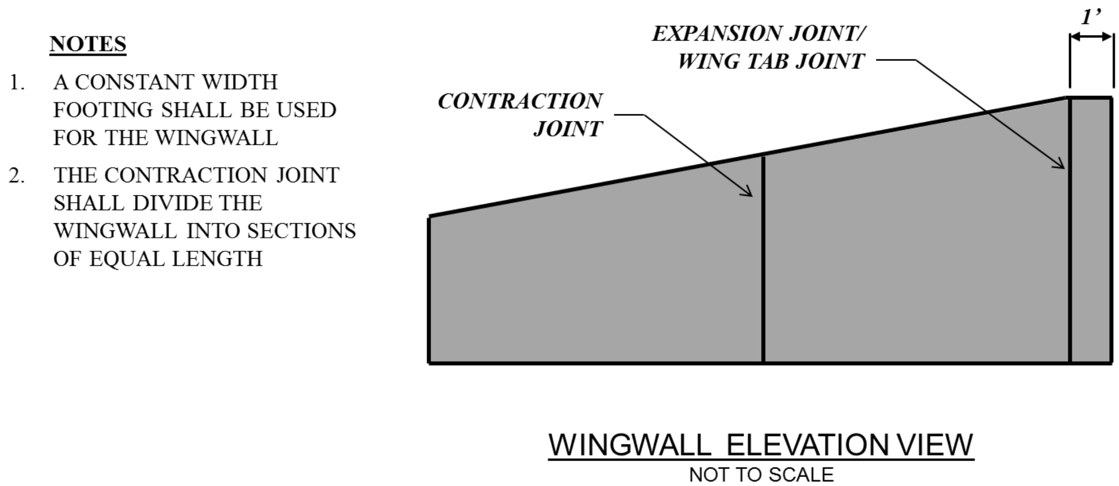


Figure 6-3: Wingwall Elevation View (Not to Scale)

6.2.2.1 Expansion Joint

A wingwall expansion joint was designed to alleviate stresses from the thermal expansion of concrete, and to allow for rotation at the culvert/wingwall intersection if foundation settlement developed. The detail of the proposed expansion joint is shown in Figure 6-4.

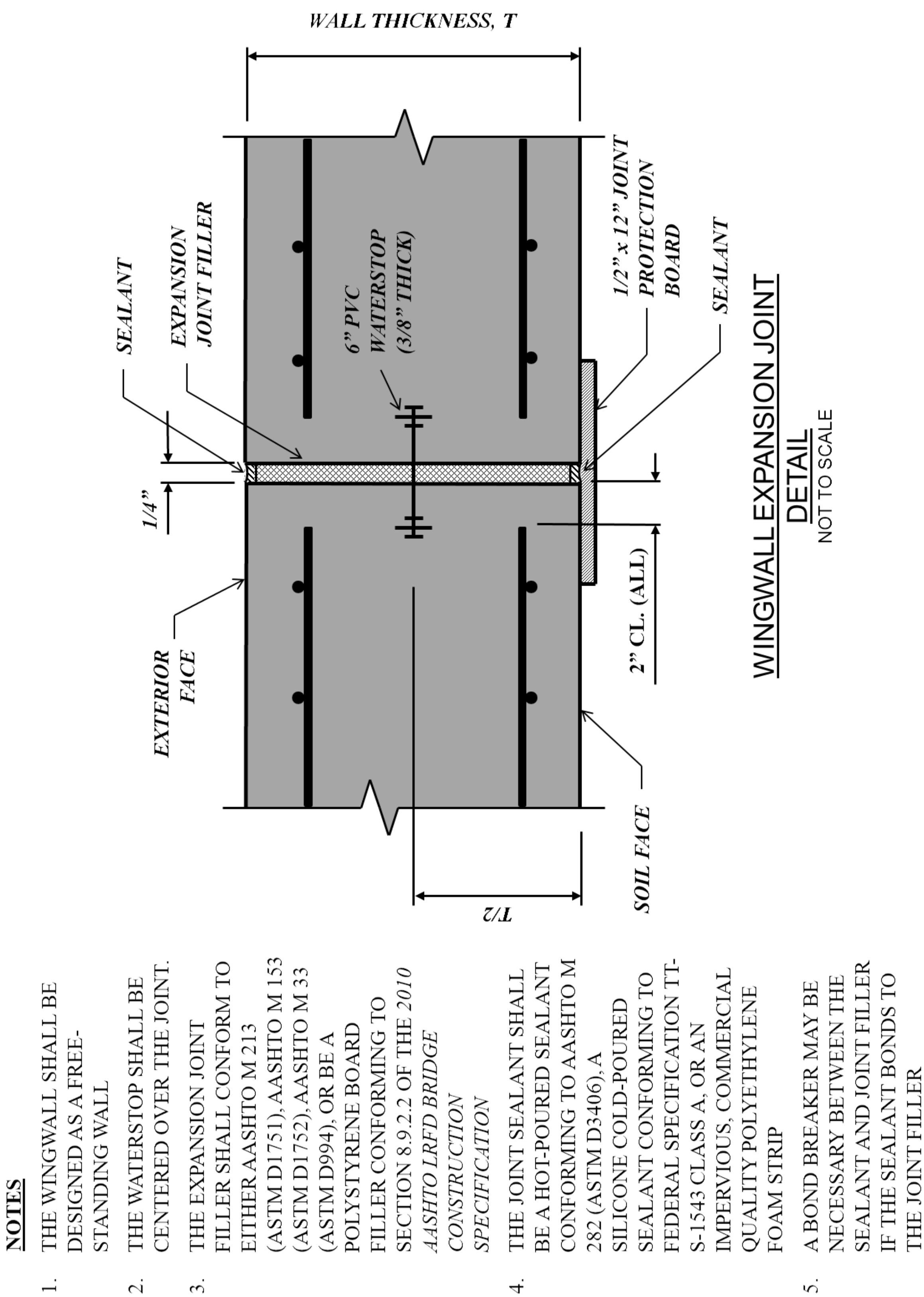


Figure 6-4: Wingwall Expansion Joint (Not to Scale)

The joint protection board and waterstop are consistent with what is proposed for the contraction joint in this thesis as shown in Figure 6-2. The aforementioned features are discussed in Section 6.2.1.2. The expansion joint shall be located 1 ft (305 mm) from the beginning of the wingwall, as is specified by the TDOT (2000d). Refer to Figure 2-47 of this thesis for an illustration of a TDOT wingwall. As is consistent with what the TDOT (2000d) specifies, the reinforcement through the expansion joint should not be continuous. This is done to reduce restraint, and to allow the joint to move as needed. The 1/4–in. expansion joint dimension is consistent with what is specified by the LA DOTD (2008) in Figure 2-37. This dimension is used to allow thermal expansion movements to occur between the culvert and wingwall.

The wingwall is to be designed as a free-standing wall; therefore, a key will not be included in the expansion joint design, because no load transfer is required at the joint if the wall is designed as a free-standing wall. This will also allow the wingwall to be structurally stable in the event that foundation settlement occurs, and causes rotation at the intersection of the culvert and wingwall.

As required in AASHTO (2010a) Section 8.9.2, the expansion joint filler shall be a preformed structural or pavement joint filler conforming to AASHTO M 213 (ASTM D1751), a preformed sponge rubber or cork joint filler conforming to AASHTO M 153 (ASTM D1752), a preformed concrete joint filler conforming to AASHTO M 33 (ASTM D994), or a polystyrene board filler.

As required in AASHTO (2010a) Section 8.9.2.4, The joint sealant used shall be a hot-poured sealant conforming to AASHTO M282 (ASTM D3406), a silicone cold-poured sealant conforming to Federal Specification TT-S-1543 Class A, or an impervious commercial quality polyethylene foam strip. No backer rod will be required since the joint filler will act as backup material, and a bond breaker may be required if the sealant will bond to the joint filler (ACI 504 1990).

The waterstop used is the same as is used in the proposed transverse contraction joint in Section 6.2.1. It is discussed in Section 6.2.1.2 of this thesis. The use of a waterstop at this location is necessary to prevent water from moving through the joint, which may lead to erosion of the material behind the wall.

6.2.2.2 Wing Tab Joint

The wing tab joint was designed by ALDOT (2011b) for use in the Birmingham Northern Beltline Project. The wing tab joint is shown below in Figure 6-5.

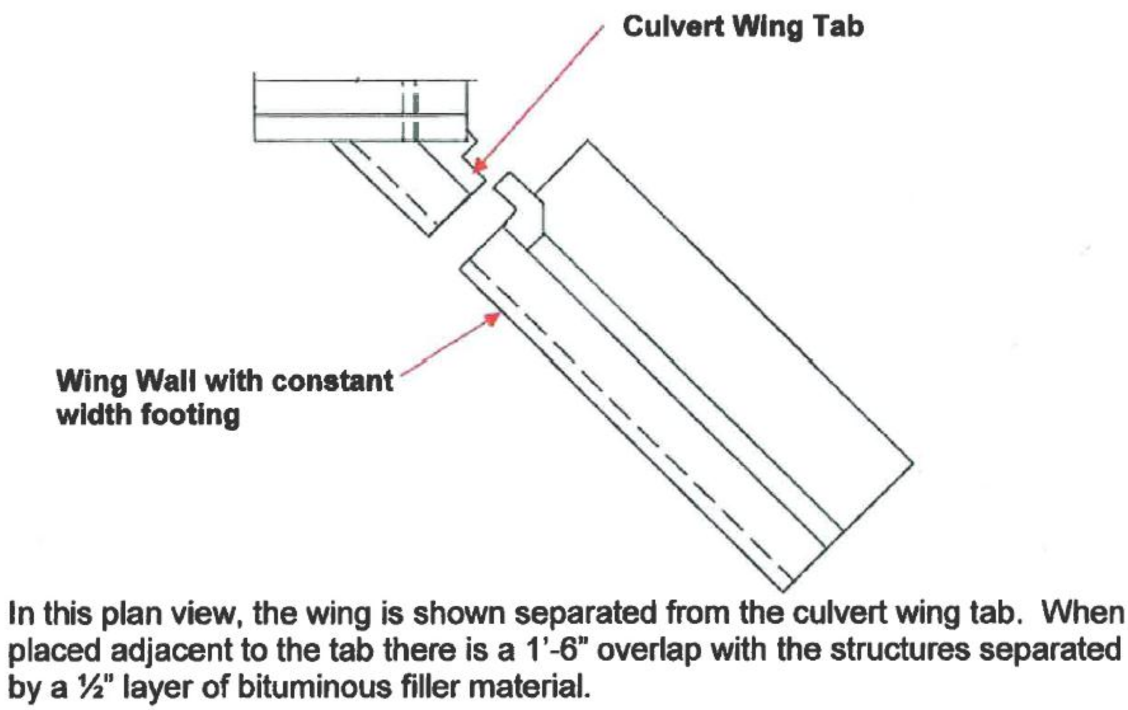


Figure 6-5: Wing Tab Joint (ALDOT 2012a)

The wing tab joint, like the proposed expansion joint, is designed to allow free rotation and torsion at the culvert/wingwall intersection if foundation settlement occurs. This joint allows for shear transfer between the wingwall and supporting culvert wall as well. The downside is that intricate formwork will be required to construct the wing tabs, which will increase the cost of construction.

6.2.2.3 Contraction Joint

The wingwall contraction joint shall have the same detail as the transverse contraction joint in Figure 6-2. Contraction joint spacing in wingwalls should follow the same rules as those shown in Figure 6-1 for the transverse contraction joints in the rest of the culvert. If the wingwall length is less than the maximum contraction joint spacing, no contraction joint is required. If the wingwall length is greater the maximum contraction joint spacing, contraction joints must be placed so that the wingwall is divided into approximately equal sections.

If the expansion joint proposal is used, all sections of the wingwall, including the ones separated by contraction joints, must be built as free-standing walls, and a constant width foundation shall be used.

6.3 Summary

In this chapter, joint details were proposed with the purpose of mitigating the effects of the restrained concrete volume change cracking that has been experienced in ALDOT CIP reinforced concrete box culverts. Transverse contraction joints to be used in the barrel of the culvert were proposed in this section, as well as joints to be used in culvert wingwalls.

A keyed transverse contraction joint was proposed to allow for joint movement between culvert barrel sections. These joints shall be spaced at three time times the clear height of the culvert but not greater than 36 ft (11 m) or less than 24 ft (7.3 m).

Two options were proposed to be used at the culvert/wingwall intersection. This was because of the uncertainty of the cause of the wingwall distress observed in Chapter 3. The first proposal was an expansion joint. This design called for the wingwall to be designed as a free-standing wall as a load transfer mechanism. Designing the wall to stand on its own would allow for the wingwall to be structurally stable even if rotation at the joint occurred. This joint proposal will be easy to construct and inexpensive. The second proposal was for a wing tab joint to be used. This joint was designed by ALDOT. It also allows for load transfer and for rotation at the culvert/wingwall intersection if foundation settlement occurs. However, it requires intricate formwork and will be expensive.

A vertical contraction joint for use in the wingwall was also proposed in this section. The joint design, geometry, and spacing are the same as the proposed transverse contraction joint in the rest of the culvert.

Chapter 7

Summary, Conclusions, and Recommendations

7.1 Summary

Significant cracking was observed in Alabama Department of Transportation (ALDOT) cast-in-place (CIP) reinforced concrete box culverts in the Anniston East Bypass (AEB) project. Numerous wide transverse cracks were observed inside the culvert barrels. Many of these cracks were wider than the ACI 224 (2001) limit of 0.012 in. (0.30 mm), above which cracks are considered detrimental to the structure.

Because of the cracking problems in the AEB, it was decided to perform crack condition surveys of other CIP reinforced concrete box culverts in Alabama. The crack condition surveys were done to obtain information regarding the distress discovered in culverts throughout the state. They were also done to determine if the distress observed in the AEB was unique. The majority of the transverse cracks observed in the crack condition surveys were through cracks located in the ceiling and in the walls. Few transverse cracks were observed in the base, but when discovered, they tended to be very wide. Significant cracking was also observed in the wingwalls of the culverts. Distress was found in the wingwall itself and at the intersection of the culvert and wingwall. The majority of the culverts surveyed used transverse contraction joints with continuous longitudinal reinforcement; however, one culvert in the AEB project and two in the Corridor X project used transverse contraction joints. From the data gathered in the crack condition surveys, the average crack width, average crack spacing, and 90th percentile crack width were plotted for every culvert surveyed. This data led to the finding that using contraction joints reduced the average crack widths in the AEB and Corridor X projects when compared to culverts in the same project locations that used transverse construction joints with continuous longitudinal reinforcement.

An analysis procedure, from Gilbert (1992), was used to determine the amount of temperature and shrinkage reinforcement necessary to keep the average crack width of a CIP reinforced concrete box culvert wall at or below the ACI 224 (2001) limit of 0.012 in. (0.30 mm). The analysis procedure was modified by the author to include movement at contraction joints and to include thermal and drying shrinkage.

An instrumentation and testing plan was developed for a CIP reinforced concrete box culvert under construction. The plan included obtaining concrete stress, strain, and temperature data during early ages of the concrete. It also included monitoring the crack width development of transverse cracks in the culvert. The tests included in the plan consisted of creep testing, drying shrinkage testing, tensile strength development, modulus of elasticity development, compressive strength development, maturity, setting time, quality control testing, and early-age restrained stress development. The data and results from the instrumentation and testing plan are not included in this thesis, and will be included in future research.

Transverse contraction joints and vertical wingwall joints were also developed and proposed with the purpose of mitigating cracking in ALDOT CIP reinforced concrete box culverts.

7.2 Conclusions and Recommendations

The main objectives of this research were to determine the extent of the distress in other CIP reinforced concrete box culverts in Alabama, develop an instrumentation plan to assess the stress development in a newly constructed culvert, determine the mechanism that caused cracking to occur, and to develop methods for mitigating cracking. All of these objectives were accomplished through this research effort. The instrumentation plan is mentioned in Section 7.1.

7.2.1 Cracking in Alabama Box Culverts

Similar cracking to what was observed in the AEB project was also found in the other culverts surveyed. Six of the eight culverts surveyed that were not in the AEB project had average crack widths greater than 0.012 in (0.30 mm) limit. It was concluded that the distress was not unique to the AEB project

7.2.2 Causes of Cracking

From the research data, the main cause of the transverse cracking is thought to be restrained volumetric changes in the concrete due to the combined effect of thermal and drying shrinkage. Thermal and drying shrinkage are unavoidable in concrete culverts due to temperature and thermal cycles (Schindler 2002; Hansen and Almudaiheem 1987). The very stiff base restrains the walls and ceiling of the culvert when it undergoes thermal and drying shrinkage. This causes stresses to rise and cracking to occur. This conclusion is supported by the fact that transverse contraction joints used in AEB and Corridor X culverts mitigated the distress experienced. It is also supported by through cracks indicating that thermal shrinkage is a major cause of cracking (Bernander 1998).

The source of the wingwall cracking was narrowed to two possible causes. It could be due to thermal and drying shrinkage effects, or, as suggested by ALDOT, it could be attributed to foundation settlement. The rigid wingwall footing provides restraint when the wingwall undergoes shrinkage. This restraint can lead to cracks in the wingwall itself; it can also lead to cracking at the intersection of the culvert and wingwall when the wingwall footing restrains the long culvert from shrinking. Foundation settlement can cause the wingwall to rotate and crack where the culvert and wingwall interact. Settlement can also cause a lack of support under the middle of the wingwall and cause cracking in the wingwall itself.

7.2.3 Mitigation of Cracking

Through the modified Gilbert (1992) analysis, it was determined that the temperature and shrinkage reinforcement currently used by ALDOT in CIP reinforced concrete box culverts should be increased. The analysis recommended that a reinforcement ratio of 0.0045 be used for culverts with a joint spacing of 3 times the clear height of the culvert. This value was reasonable when compared to other temperature and shrinkage reinforcement recommendations for restrained members. However, the reinforcement stresses in the analysis were higher than the yield strength of the reinforcement. These stress values were not representative of the actual stresses in the temperature and shrinkage reinforcement, but it raises questions about the analysis results. A temperature and shrinkage reinforcement ratio of 0.0040 would be economical and sufficient; however, additional finite element analysis and subsequent verification based on field measured behavior is necessary to verify this recommendation.

To relieve the thermal stresses in the culverts, it was proposed that transverse contraction joints be placed in the culverts at intervals of 3 times the clear height of the culvert, but not greater than 36 ft (11 m) or less than 24 ft (7.3 m). Culverts in the AEB and Corridor X projects that had transverse contraction joints had smaller average crack widths when compared to culverts in the same project location that used construction joints with continuous reinforcement through them.

Vertical joints were also proposed to alleviate the distress experienced in culvert wingwalls. Two proposals were made regarding the joint to be used at the intersection of the wingwall and the culvert. The first proposal was an expansion joint with the wingwall designed as a free-standing wall. Designing the wingwall to be free standing is a load-transfer mechanism, but it also allows the wingwall to be structurally stable if foundation settlement causes rotation at the intersection off the culvert and wingwall. This option would be easy to construct and relatively inexpensive. The second option would use a wing tab designed by ALDOT. This joint would also allow for load transfer and it would provide for the rotation of the wingwall at the joint if foundation settlement occurs. However, this joint would require intricate formwork and would be somewhat costly. A vertical contraction joint was also proposed for use in wingwalls. This joint would use the same detail and spacing rules as specified for the transverse contraction join in the he rest of the culvert.

7.3 Recommendations for Future Work

After completing the requirements for this research effort, there are some recommendations to be made for future work. The implementation and testing plan from Chapter 5 of this thesis should be used for a CIP reinforced concrete box culvert that is under construction. There is currently a culvert in the Birmingham Northern Beltline project that is designated to be instrumented by Auburn University. This will increase the knowledge of the effects that temperature has on the concrete properties in box culverts and the stress development that corresponds to the temperature. The data from the instrumentation and testing plan will allow for other mitigation measures to be taken in culverts. The effects of the amount of temperature and shrinkage reinforcement used in CIP reinforced concrete box culverts should also be further investigated. While this topic was referenced in this thesis, finite element analysis would provide a more accurate recommendation for the amount of temperature and shrinkage reinforcement to be used.

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Appendix A

Box Culvert Crack Condition Survey Data and Data Sheet

The crack width and crack spacing histograms for each culvert surveyed and the data sheet used when performing the culvert crack condition surveys in Chapter 3 is presented in Appendix A.

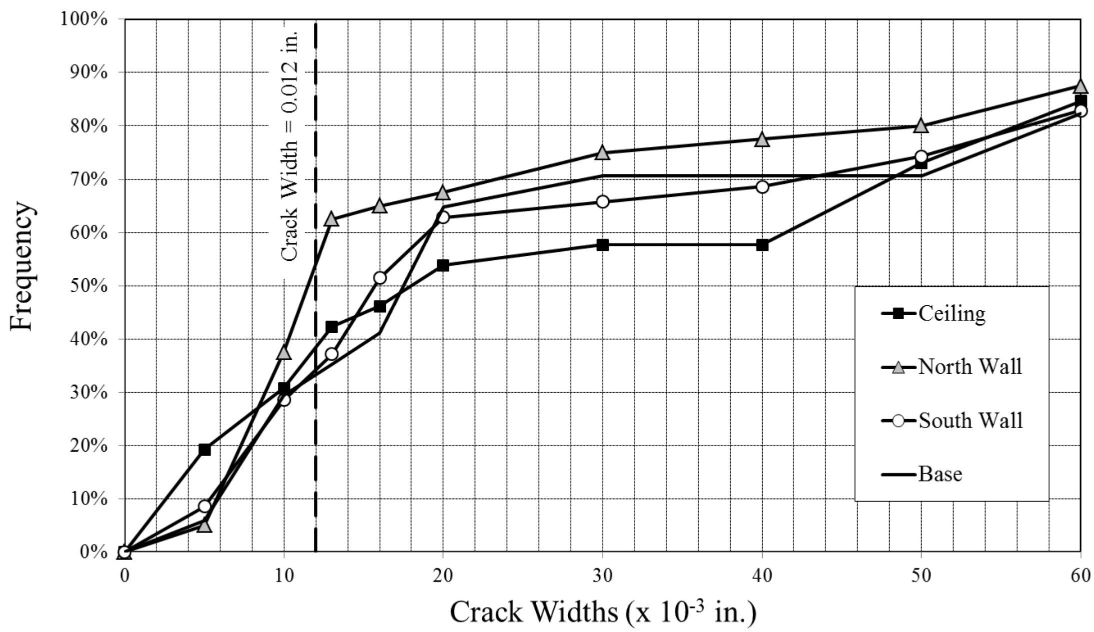

Figure A-1: AEB Culvert at 175+70 Crack Width Histogram

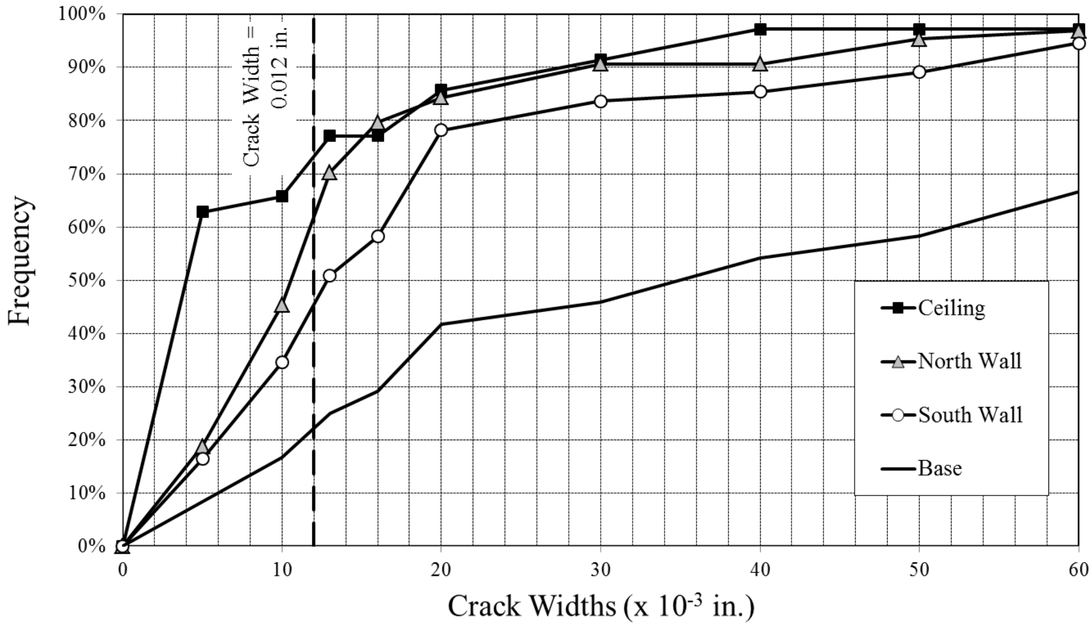

Figure A-2: AEB Culvert at 162+90 Crack Width Histogram

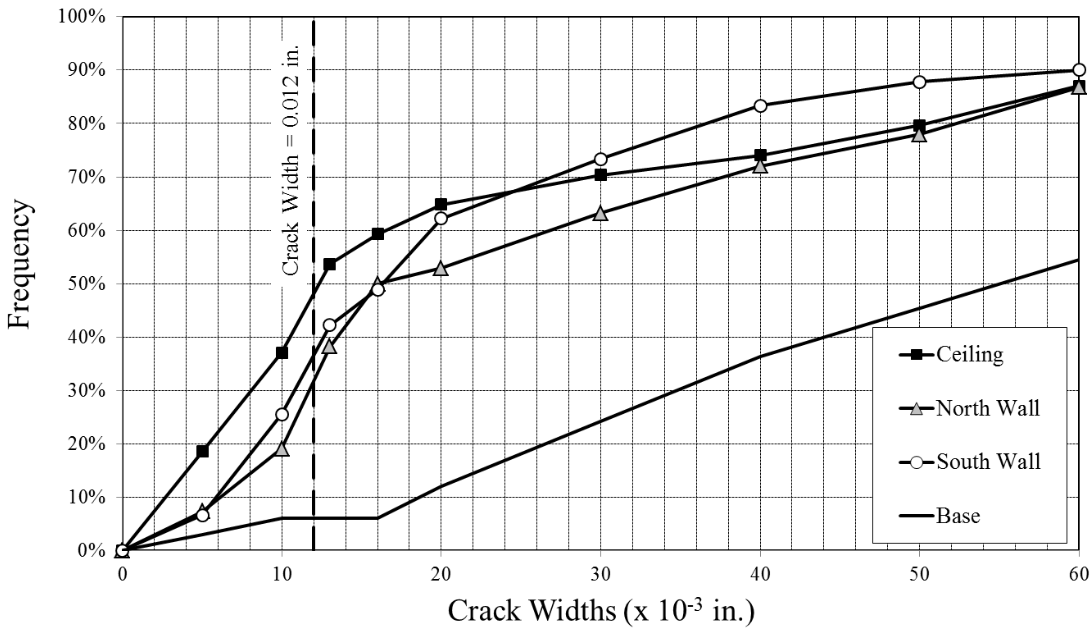

Figure A-3: AEB Culvert at 149+60 Crack Width Histogram

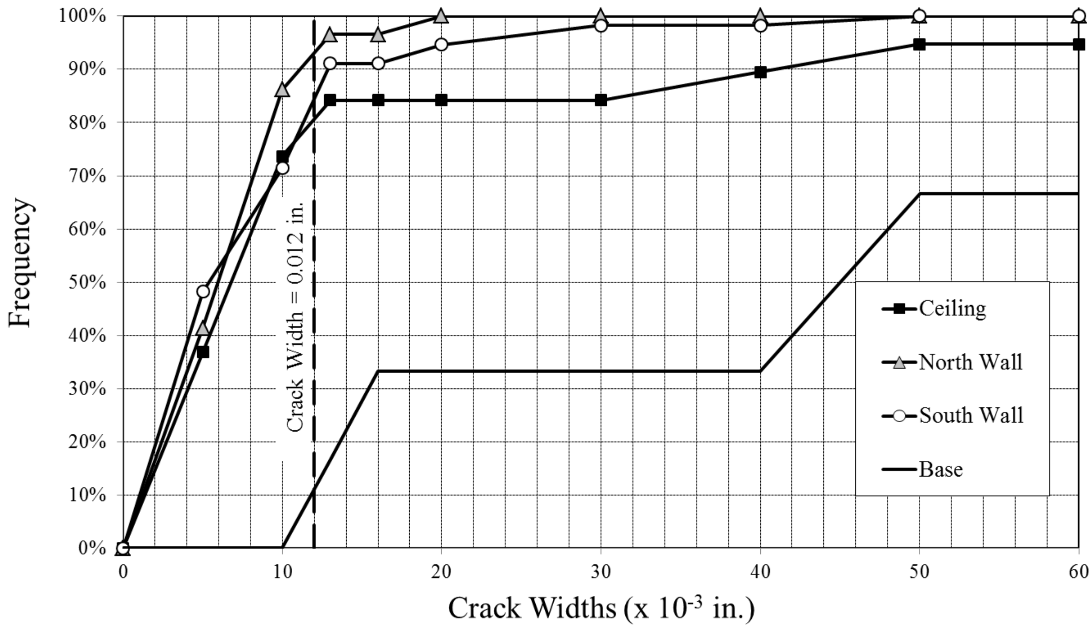

Figure A-4: AEB Culvert at 257+69 Crack Width Histogram

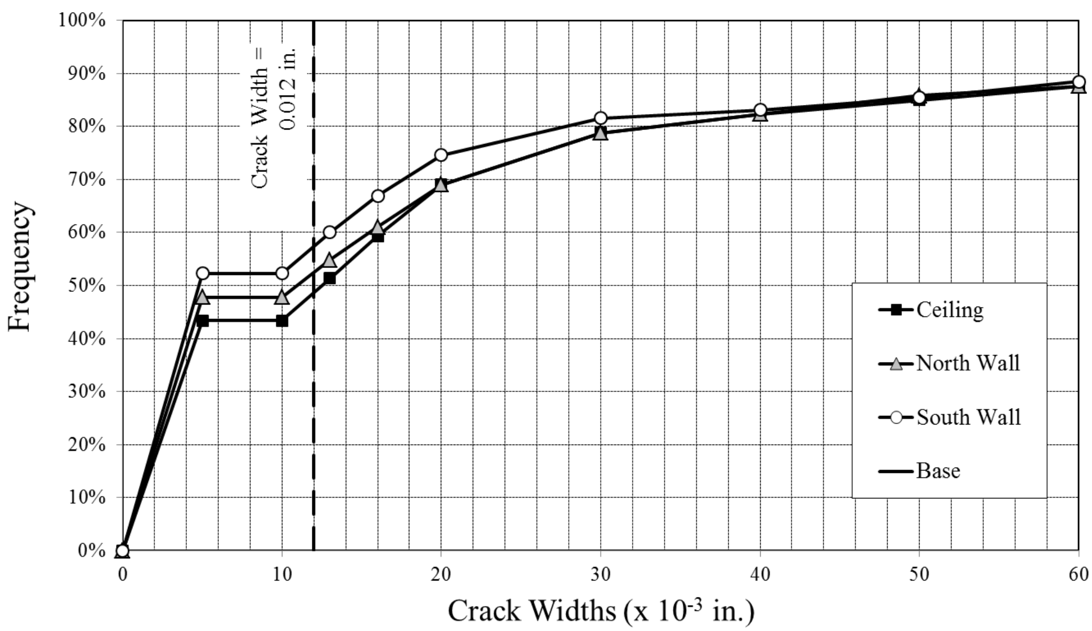

Figure A-5: AEB Culvert at 240+37 Crack Width Histogram

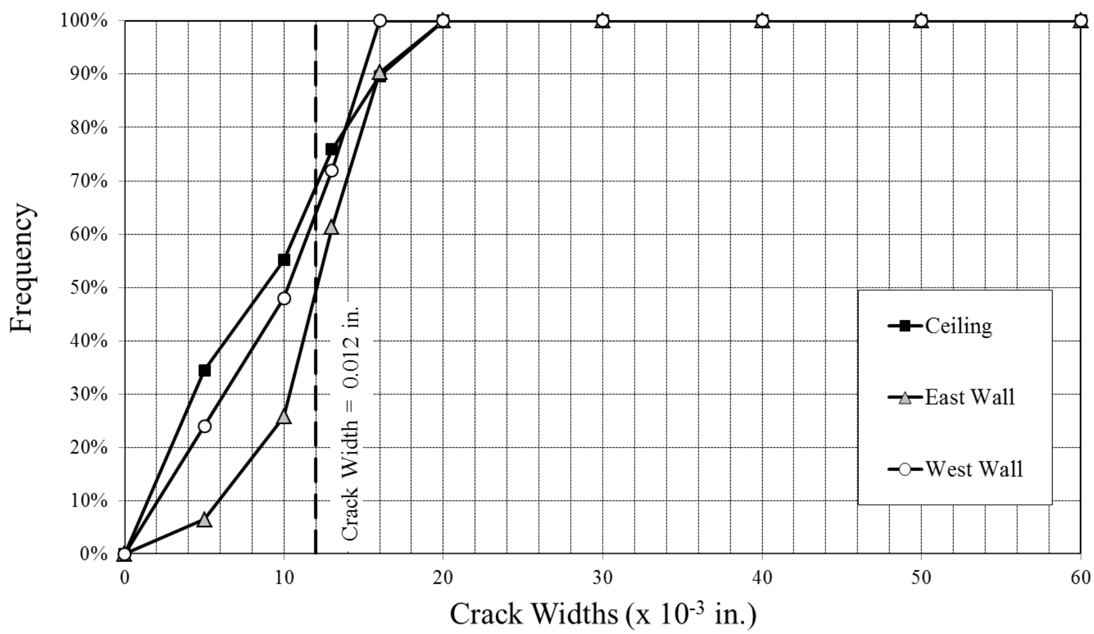

Figure A-6: Centreville Culvert at 1808+98 East Barrel Crack Width Histogram

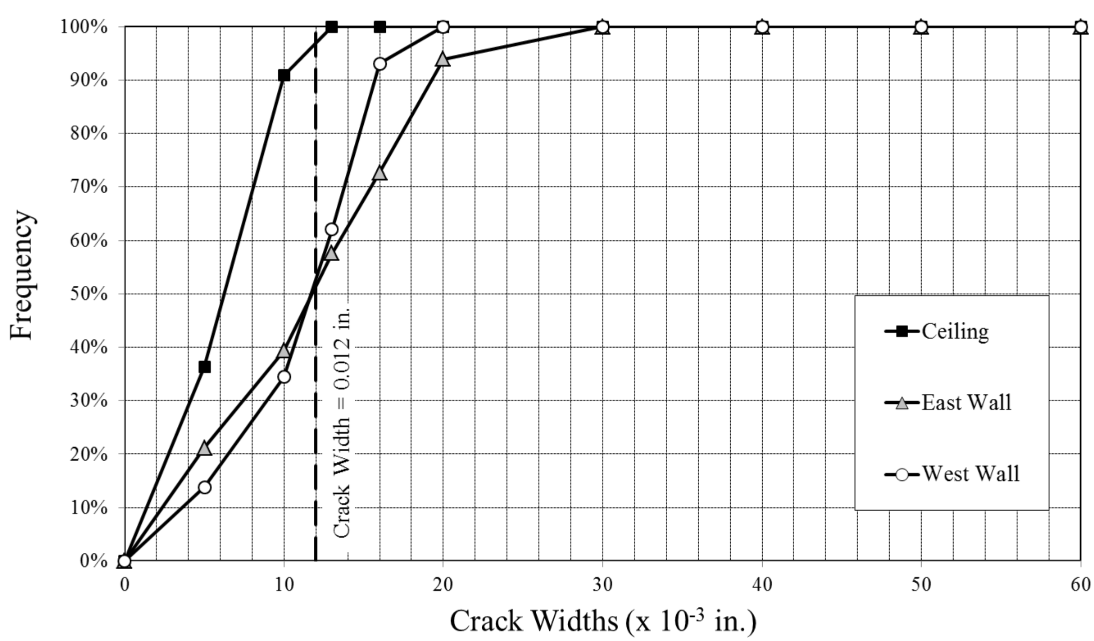

Figure A-7: Centreville Culvert at 1808+98 Center Barrel Crack Width Histogram

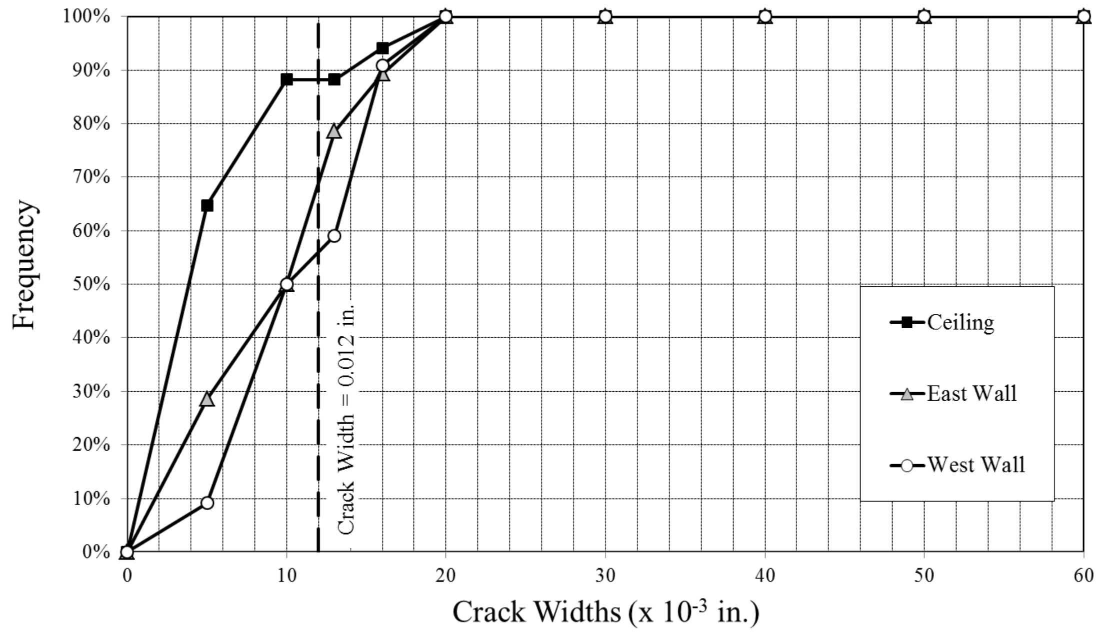

Figure A-8: Centreville Culvert at 1808+98 West Barrel Crack Width Histogram

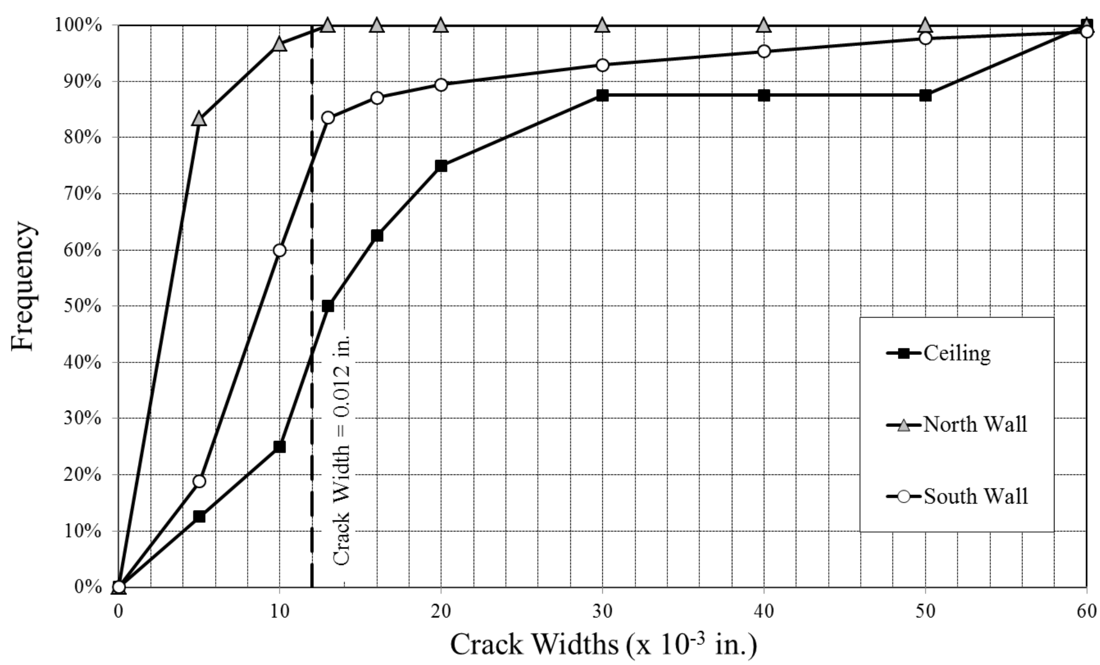

Figure A-9: Corridor X Culvert at 4877+13 Crack Width Histogram

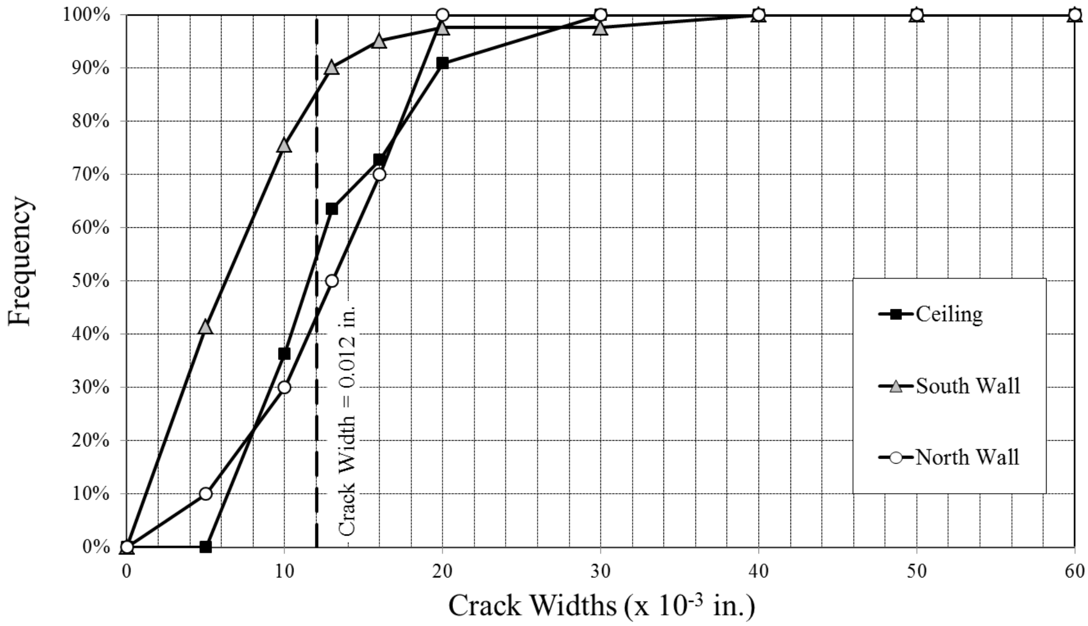

Figure A-10: Corridor X Culvert at 4959+43 Crack Width Histogram

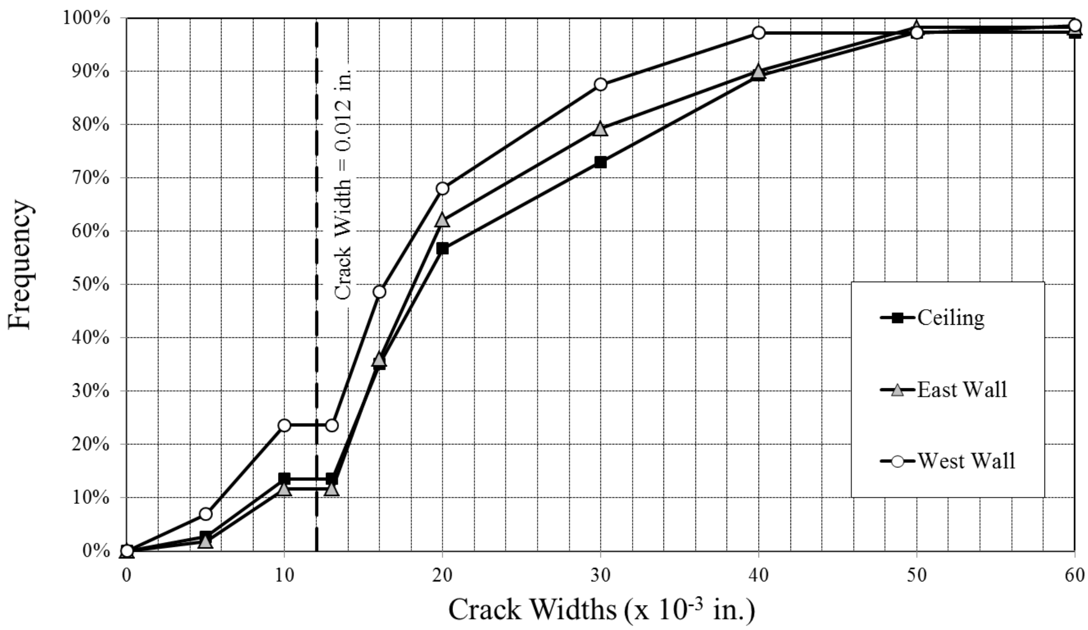

Figure A-11: Corridor X Culvert at Exit 85 Crack Width Histogram

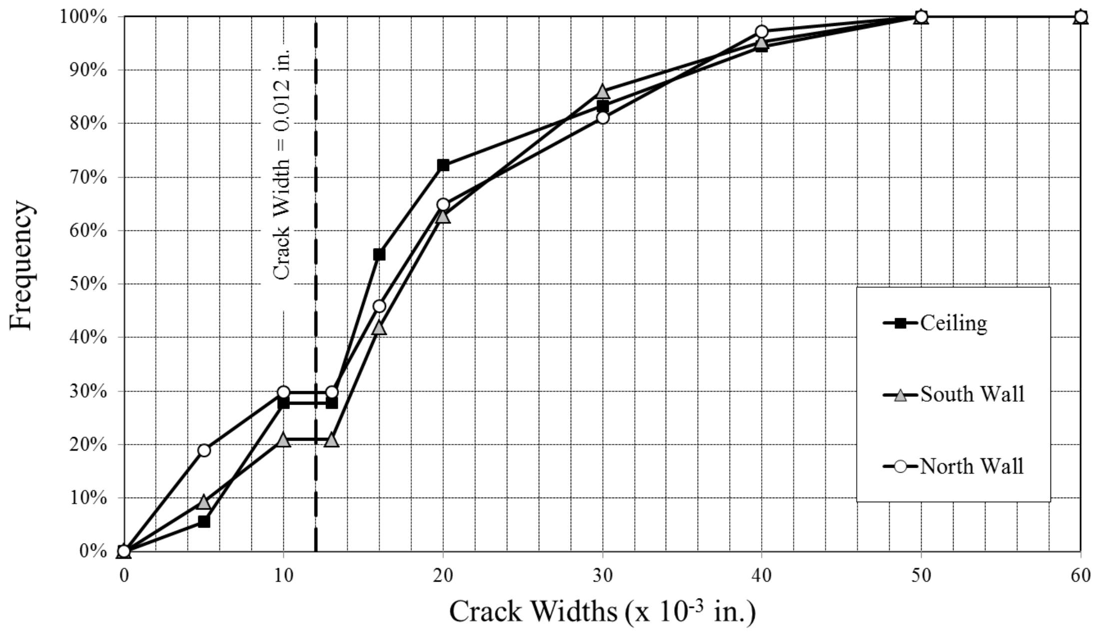

Figure A-12: Dadeville Culvert at 45+31.55 Crack Width Histogram

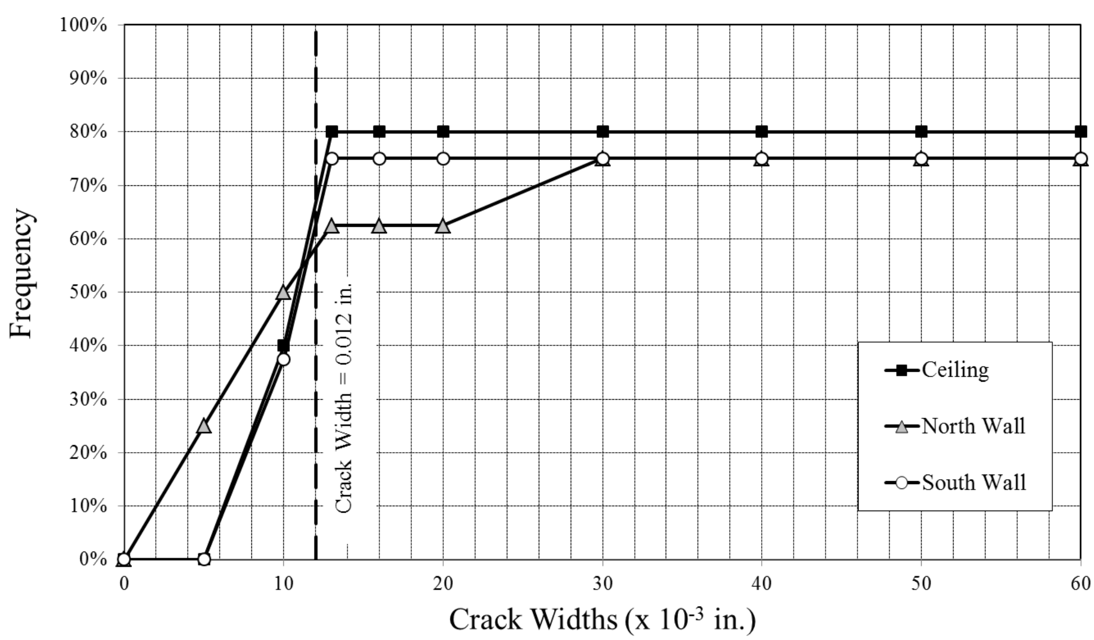

Figure A-13: Dutton Culvert at 548+23 Crack Width Histogram

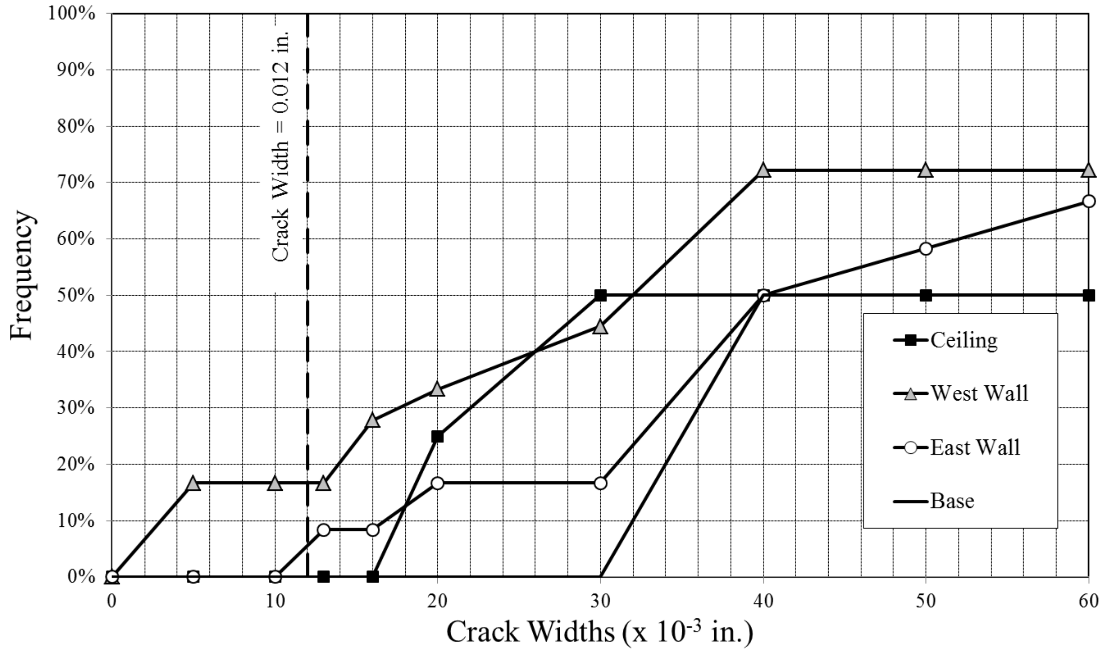

Figure A-14: Prattville Culvert on US-82 Crack Width Histogram

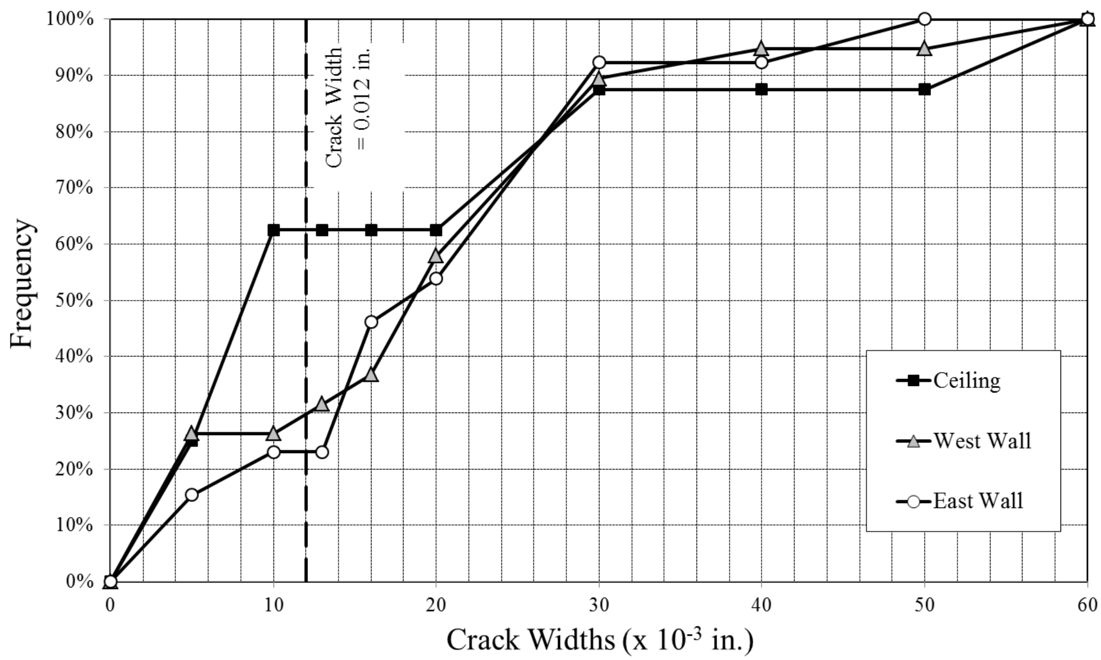

Figure A-15: I-85 North Culvert Crack Width Histogram

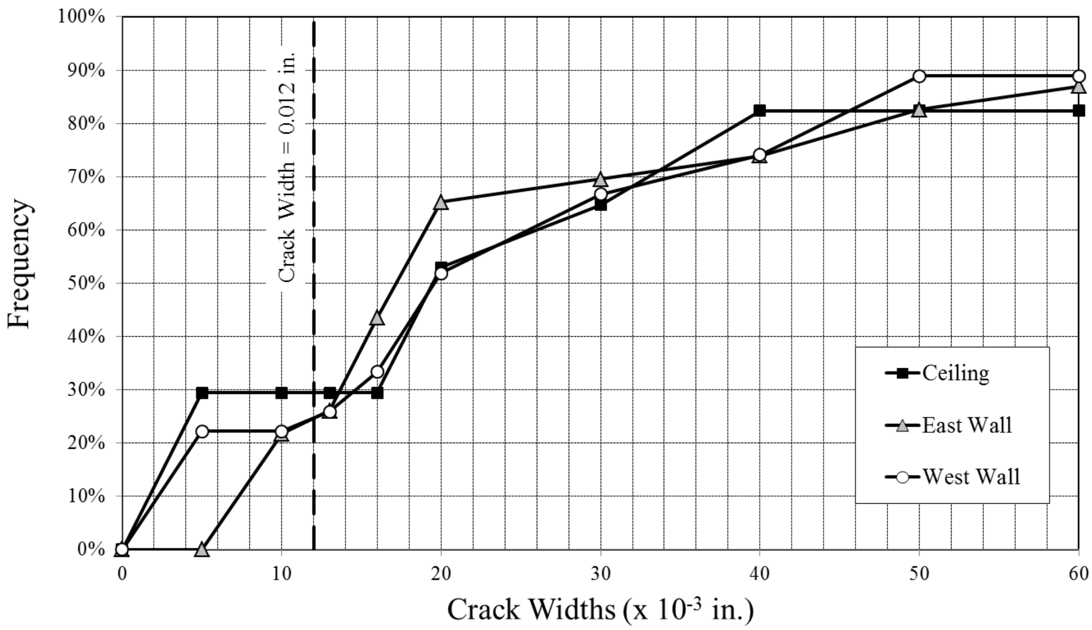

Figure A-16: I-85 South Culvert Crack Width Histogram

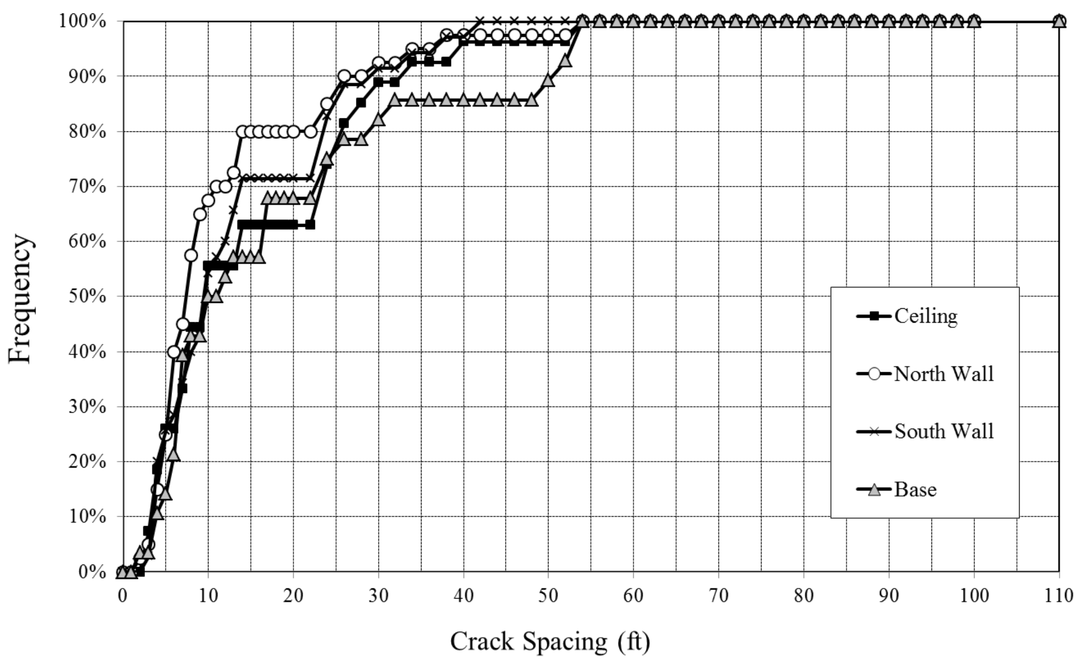

Figure A-17: AEB Culvert at 175+70 Crack Spacing Histogram

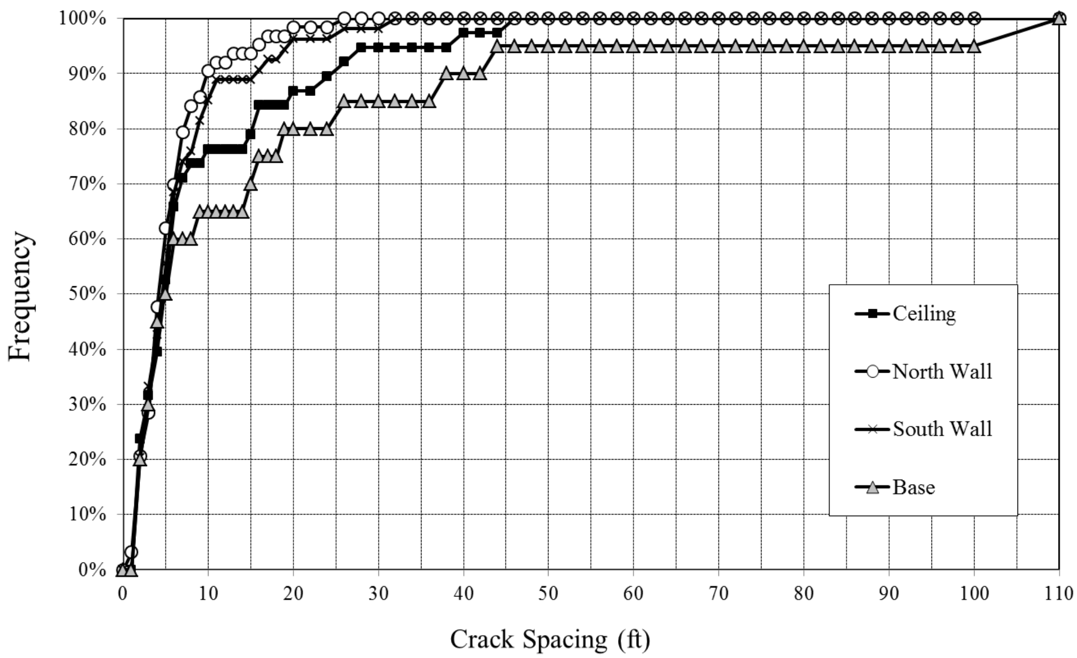

Figure A-18: AEB Culvert at 162+90 Crack Spacing Histogram

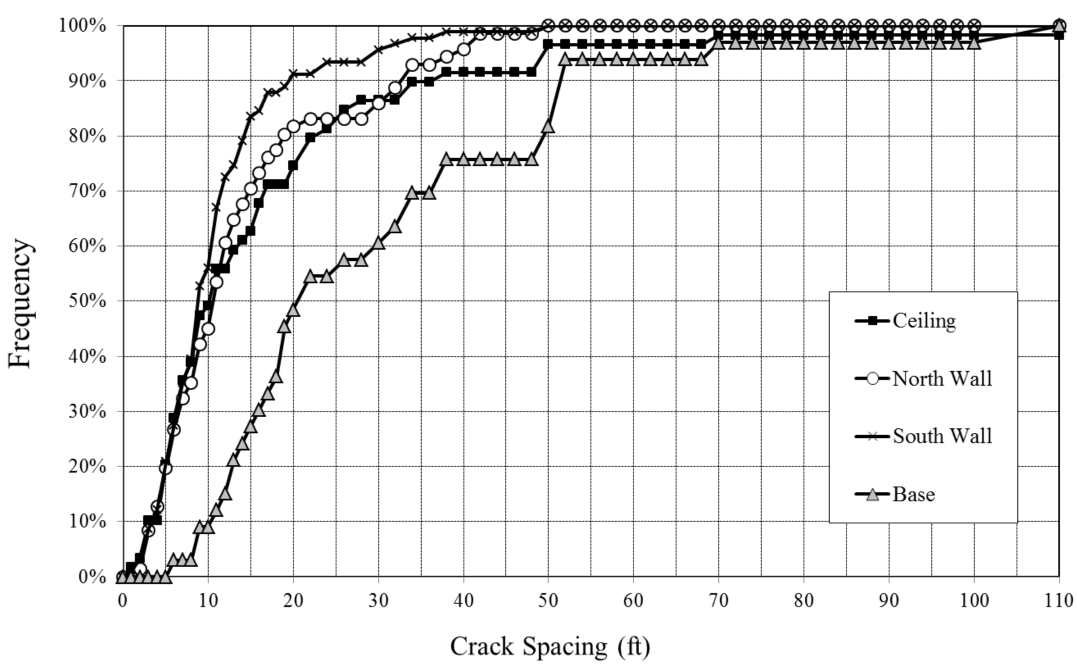

Figure A-19: AEB Culvert at 149+60 Crack Spacing Histogram

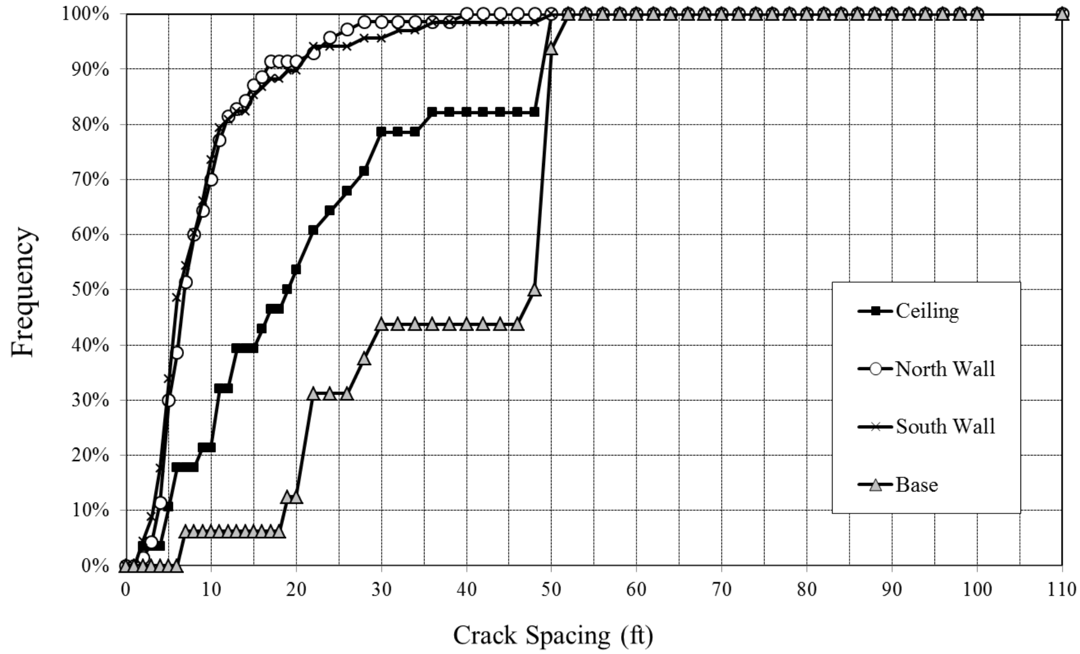

Figure A-20: AEB Culvert at 257+69 Crack Spacing Histogram

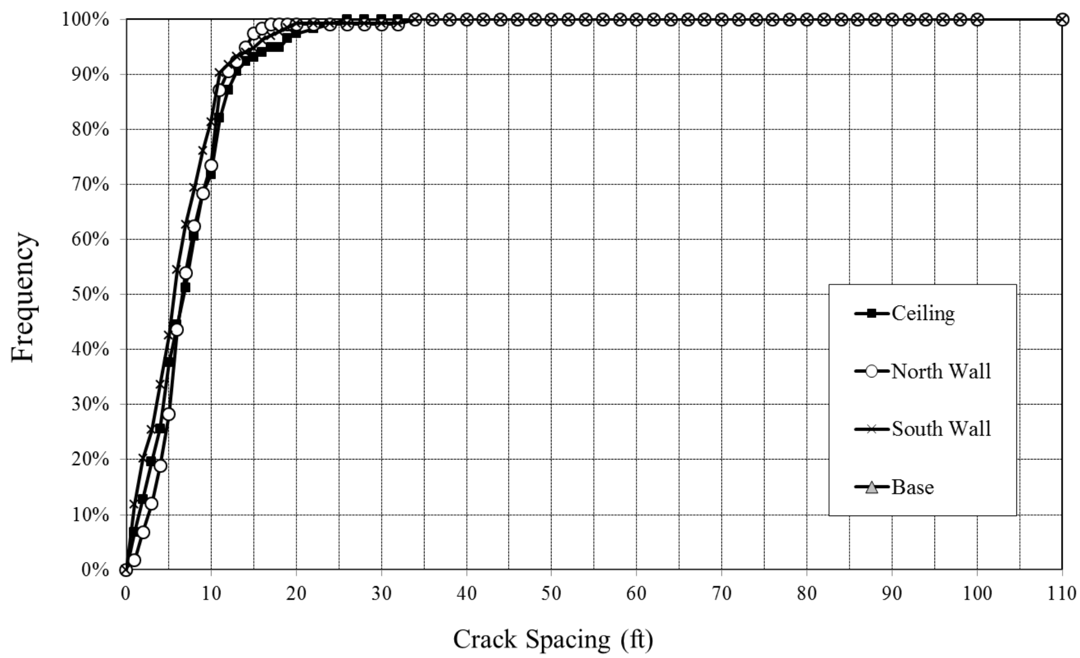

Figure A-21: AEB Culvert at 240+37 Crack Spacing Histogram

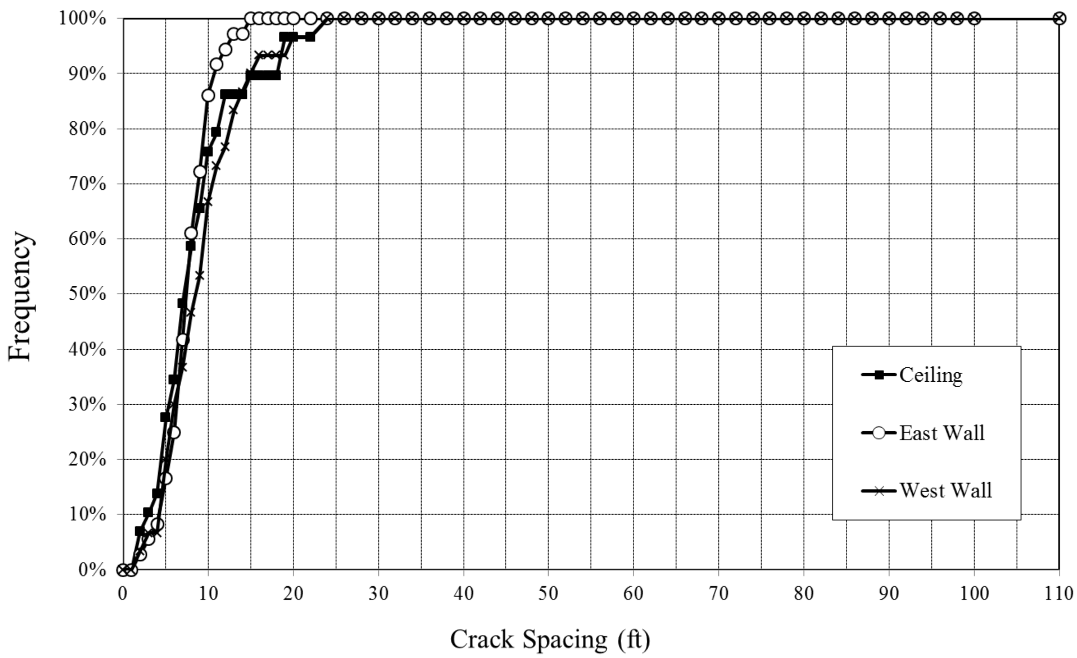

Figure A-22: Centreville Culvert at 1808+98 East Barrel Crack Spacing Histogram

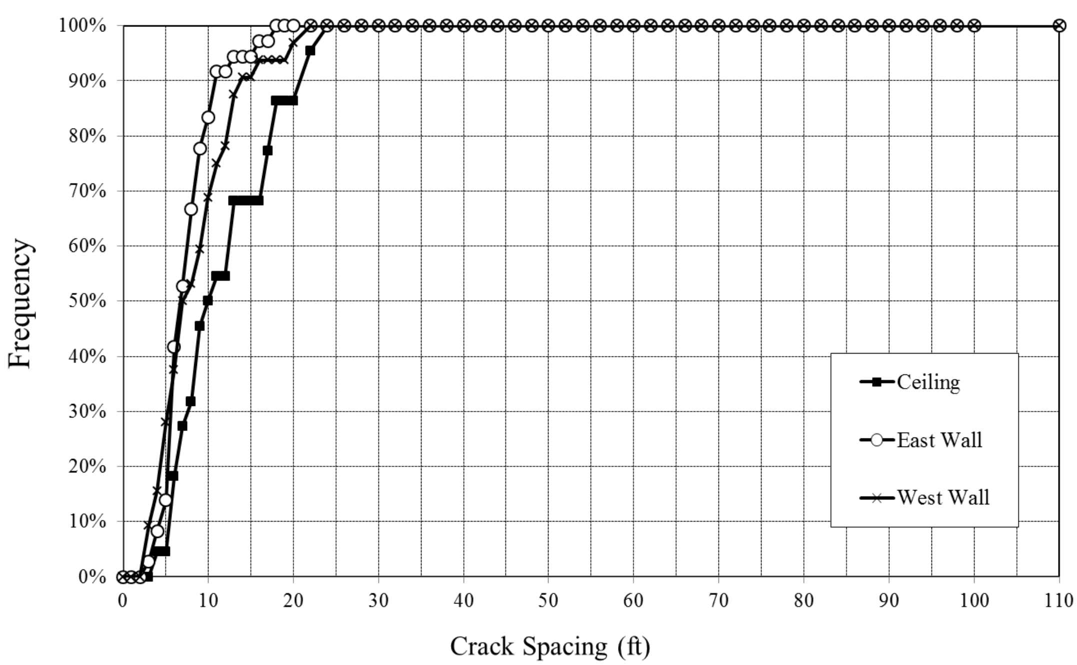

Figure A-23: Centreville Culvert at 1808+98 Center Barrel Crack Spacing Histogram

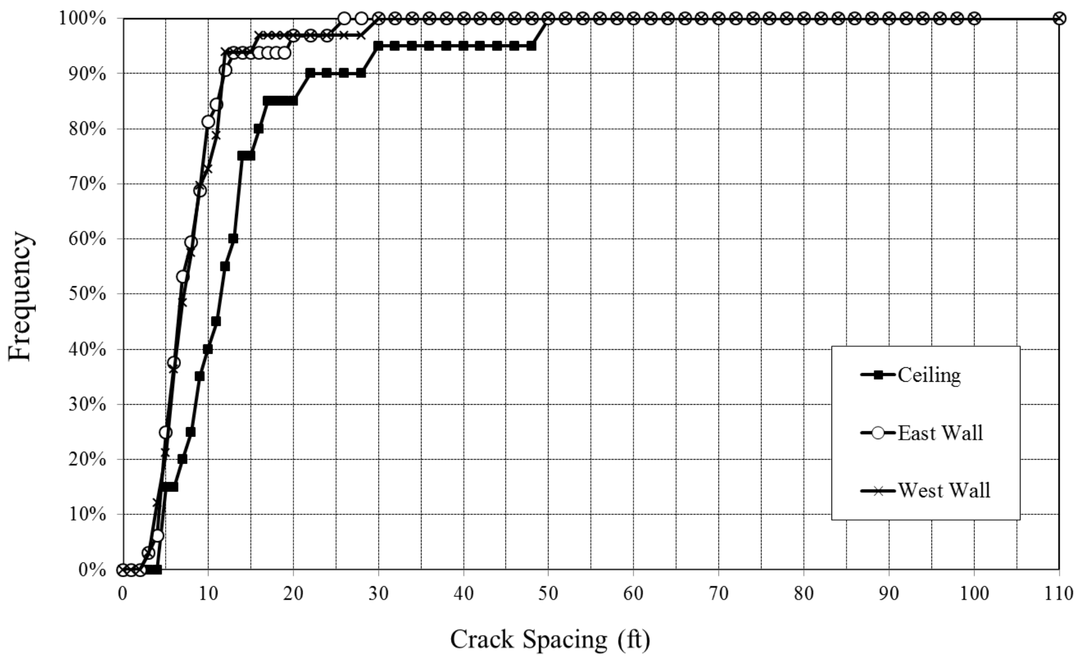

Figure A-24: Centreville Culvert at 1808+98 West Barrel Crack Spacing Histogram

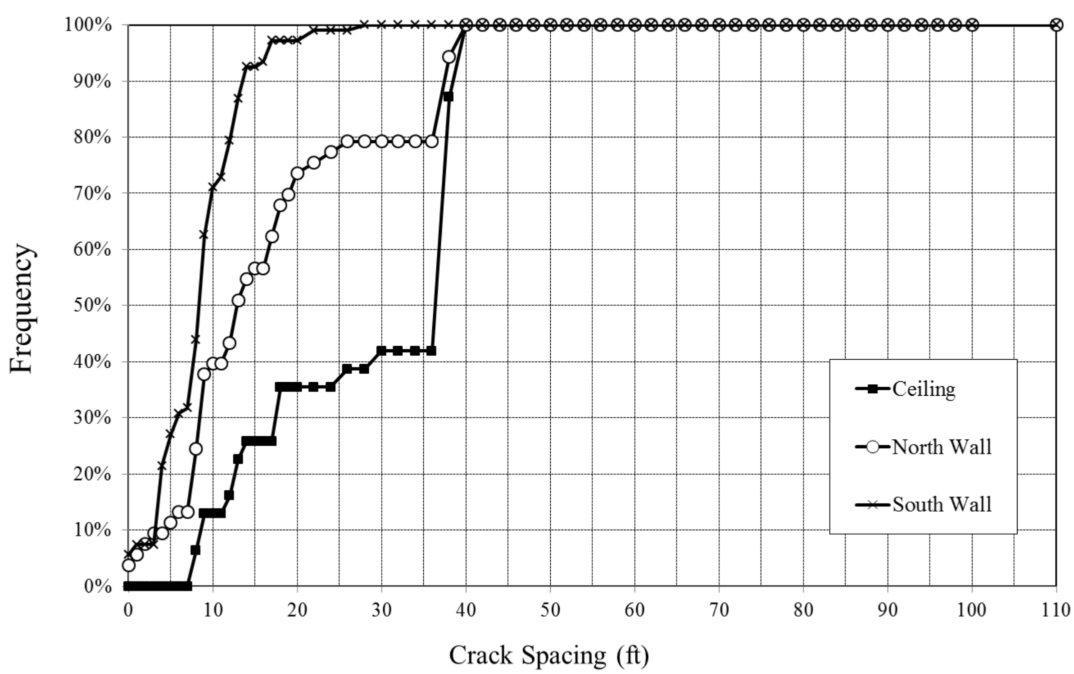

Figure A-25: Corridor X Culvert at 4877+13 Crack Spacing Histogram

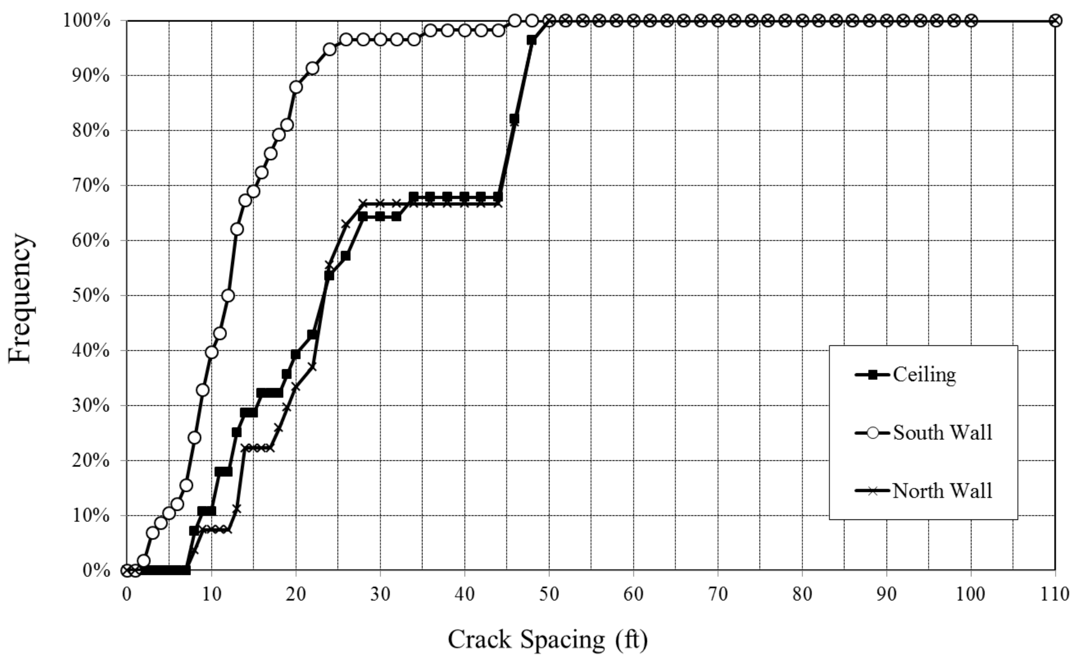

Figure A-26: Corridor X Culvert at 4959+43 Crack Spacing Histogram

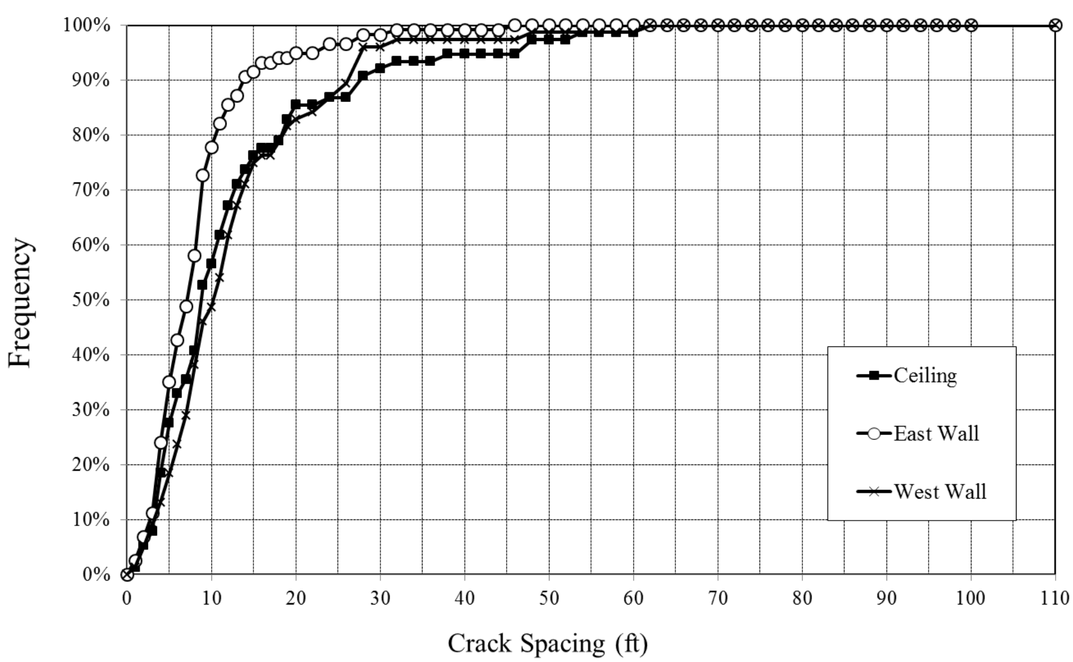

Figure A-27: Corridor X Culvert at Exit 85 Crack Spacing Histogram

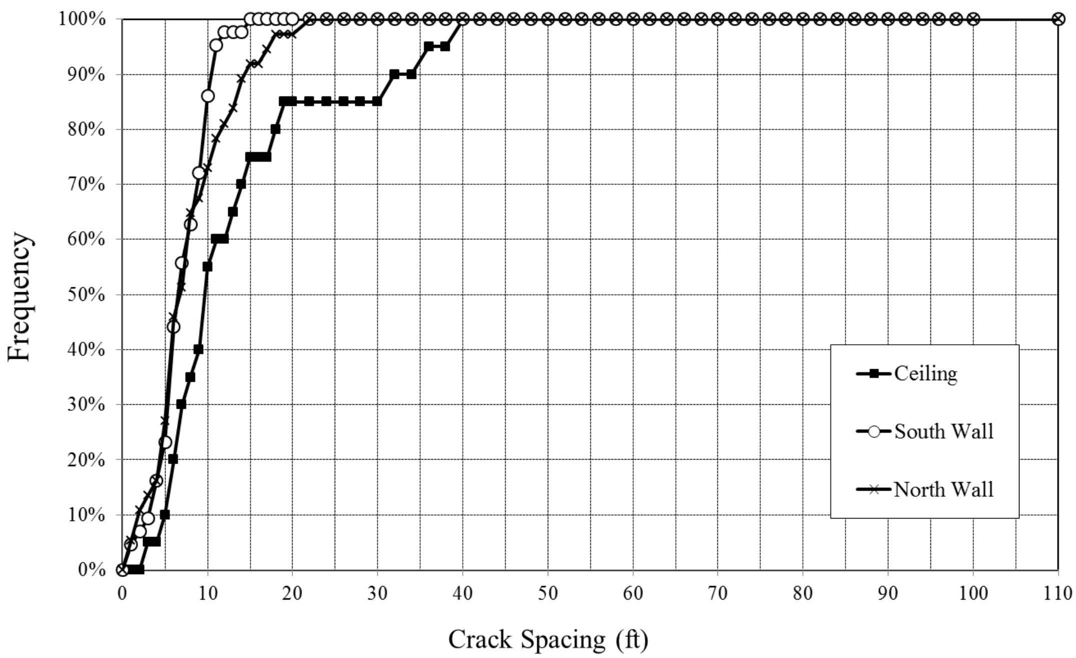

Figure A-28: Dadeville Culvert at 45+31.55 Crack Spacing Histogram

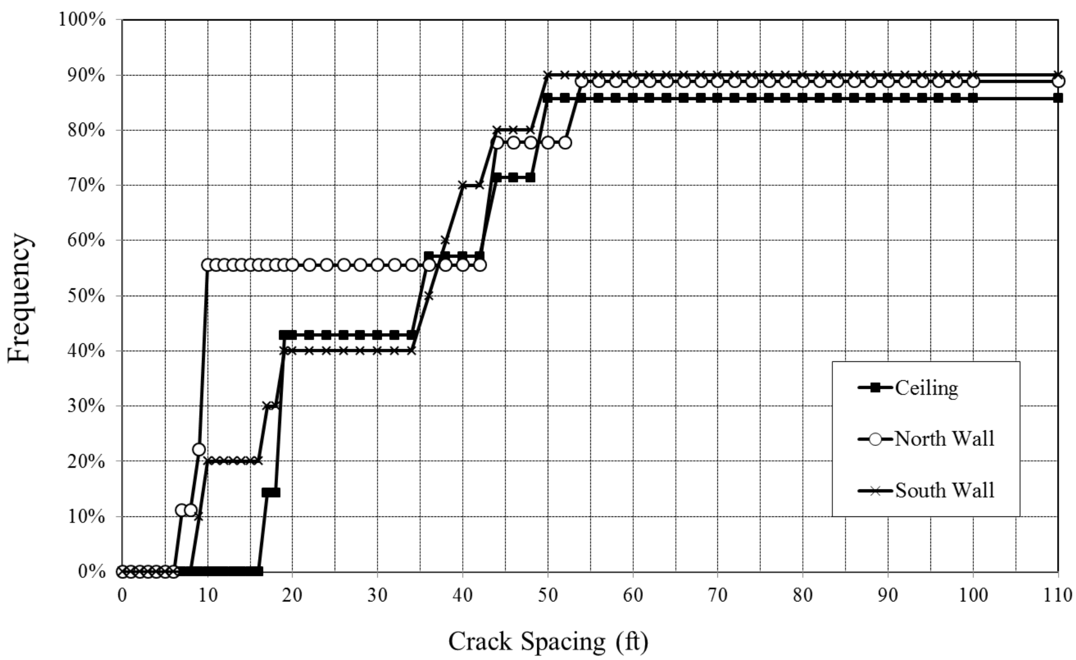

Figure A-29: Dutton Culvert at 548+23 Crack Spacing Histogram

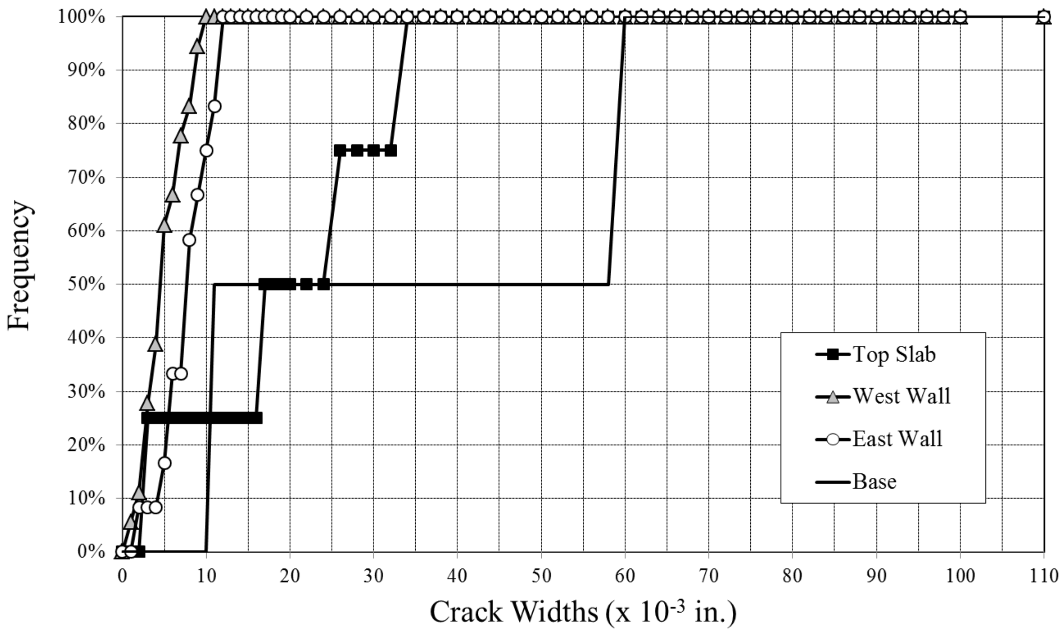

Figure A-30: Prattville Culvert on US-82 Crack Spacing Histogram

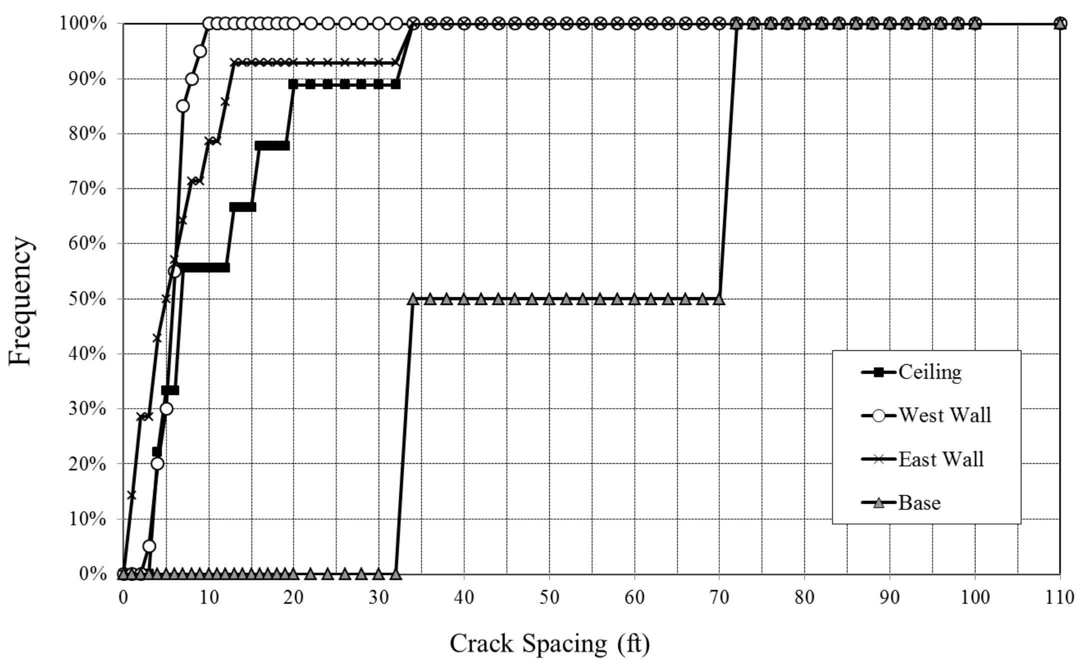

Figure A-31: I-85 North Culvert Crack Spacing Histogram

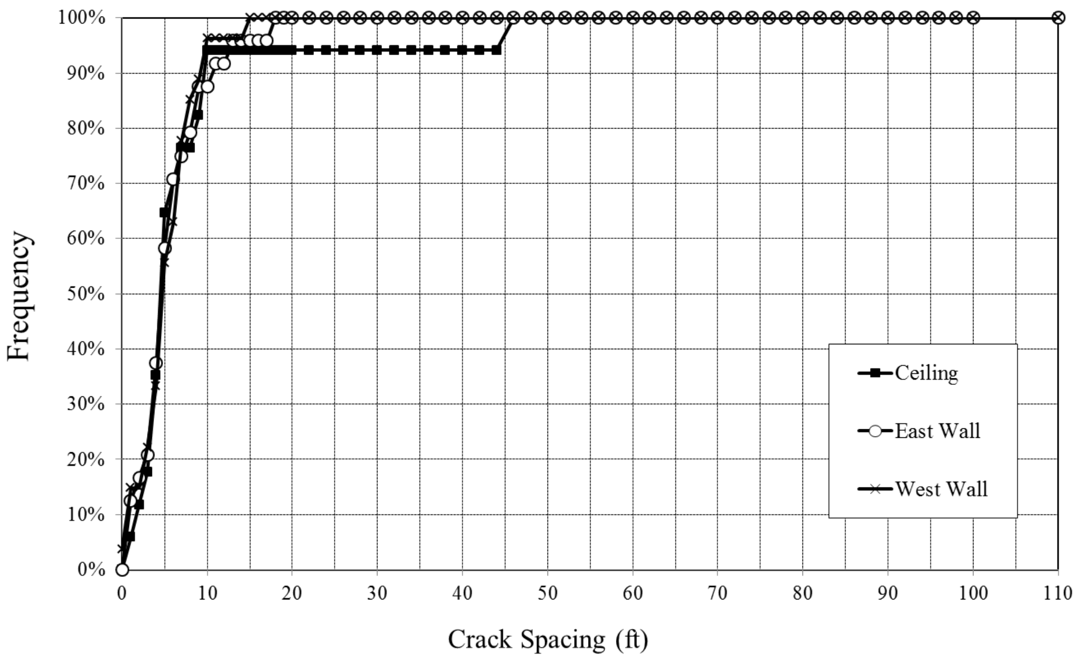

Figure A-32: I-85 South Culvert Crack Spacing Histogram

Concrete Culvert Visual Inspection Survey

General:

Inspectors: ____

Date of Survey: ____

Description of Culvert:

Culvert Identification: ____

Culvert Station: ____

Size: ____

End of Entry: ____

Environment:

Temperature: ____

Cloud Cover: ____

Precipitation: ____

Notes:____

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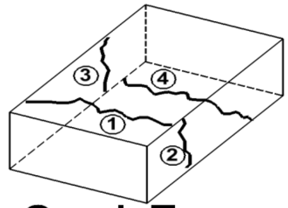
Culvert Crack Survey:



Crack Types

Station Start: ____________ Description: _______________

Station End: ___________ Description: _______________

Sta. (ft)	Crack Type	Width (in./ 1000)	Comments	Sta. (ft)	Crack Type	Width (in./ 1000)	Comments

Notes: (L) Leaching, (S) Stalactites, (HL) Hairline, (CJ) Construction Joint, (C) Corrosion, (E) Efflorescence, (PS) Plastic Shrinkage, (CM) Cannot Measure, (PR) Previously Repaired, (CON) Contraction Joint

Surface Conditions of Concrete:

Scaling	Area, Depth		
	Type		
Spalls and Popouts	No., Size, Depth		
	Type		
Stains, Efflorescence			
Exposed Reinforcement: Corrosion			
Previous Repairs			

AEB Culvert at 240+37 (J) - Dimensions - (8'x8') - ALDOT Condition Survey

STA	STA	LOCATION	WIDTH (INCHES)	WIDTH (x 10^{-3} in.)	REMARKS
11		1	0.013	13	
11		3		13	
17		1	0.005	5	
17		3		5	
21		1	0.013	13	
21		3		13	
22		3	0.013	13	
32			0.005	5	
32.5		1	0.020	20	
32.5		2		20	
32.5		3		20	
35		3	0.005	5	
36		1	0.005	5	
36		3		5	
39		2	0.005	5	
41	CONST. JOINT	1	0.030	30	GROUTED
41		2		30	
41		3		30	
41		4		30	
52		1	0.050	50	
52		2		50	
52		3		50	
53		1	0.040	40	
53		2		40	
55		1	0.013	13	
55		3		13	
60		1	0.050	50	
60		2		50	
60		3		50	
62		3	0.005	5	
66		1	0.03	30	
66		2		30	
66		3		30	
70		3	0.005	5	
71		1	0.030	30	
71		3		30	
72		1	0.040	40	
72		2		40	
80		1	0.016	16	
78		3	0.005	5	
82		3	0.005	5	
84.5		2	0.020	20	
88		1	0.005	5	
88		3		5	

STA	STA	LOCATION	WIDTH	WIDTH	REMARKS
95	CONST. JOINT	1	0.020	20	GROUTED
95		2		20	
95		3		20	
95		4		20	
33					3" CORE HOLE IN CENTER WALL
98					3" CORE HOLE IN CENTER WALL
100		2	0.005	5	
105		1	0.005	5	
105		3		5	
106		3	0.005	5	
108		2	0.005	5	
111		3	0.005	5	
117		1	0.013	13	
117		2		13	
117		3		13	
117		4		13	
123		2	0.005	5	
125		1	0.013	13	
125		2		13	
125		3		13	
128		2	0.013	13	
135		3	0.005	5	
138		1	0.016	16	
138		2		16	
144		3	0.005	5	
145.5	CONST. JOINT	1	0.050	50	
145.5		2		50	
145.5		3		50	
145.5		4		50	
151		2	0.005	5	
154					3" CORE HOLE IN CENTER WALL
154		1	0.016	16	
154		3		16	
155		2	0.060	60	
155		3		60	
157		2	0.050	50	
160.5		3	0.013	13	
161		2	0.016	16	
161		3	0.005	5	
165		1	0.030	30	
165		2		30	
165		3		30	
167		3	0.005	5	
171		2	0.016	16	
171		3		16	

STA	STA	LOCATION	WIDTH	WIDTH	REMARKS
177		2	0.030	30	
178			0.005	5	
183.5		1	0.030	30	
183.5		2		30	
183.5		3		30	
183.5		4		30	
189		2	0.005	5	
190		3	0.005	5	
194		2	0.005	5	
194		3		5	
196	CONST. JOINT	1	0.030	30	GROUTED, WATER LEAVING
196		2		30	AT FLOOR
196		3		30	
196		4		30	
197		3	0.016	16	
202		2	0.005	5	
204		1	0.016	16	
204		3		16	
207		1	0.125	125	1/8" HOLE IN FLOOR
207		2		125	LOOSING WATER
207		3		125	
207		4		125	
208		1	0.016	16	
208		3		16	
210.5		2	0.005	5	
214		1	0.020	20	
214		3		20	
216		2	0.125	125	SKEWED CRACK
216		3		125	
216		4		125	
221		1	0.070	70	
221		2		70	
221		3		70	
221		4		70	
227		3	0.005	5	
228		1	0.030	30	
228		2		30	
228		3		30	
234		1	0.005	5	
234		3		5	
239		1	0.030	30	
239		2		30	
239.5		3	0.005	5	
241		1	0.005	5	
241		3		5	

STA	STA	LOCATION	WIDTH	WIDTH	REMARKS
246	CONST. JOINT	1		CM	WELL GROUTED, EXCEPT
246		2		CM	FALLING OUT AT CEILING
246		3		CM	
246		4		CM	
247		1	0.060	60	SKEWED CRACK
247		2		60	
247		3		60	
250		1	0.070	70	
250		2		70	
250		3		70	
252		1	0.020	20	
252		3		20	
253.5		1	0.070	70	
253.5		2		70	
253.5		3		70	
255		3	0.005	5	
256.5		1	0.060	60	
256.5		3		60	
258		1	0.080	80	GROUTED
258		2		80	
258		3		80	
258		4		80	
259		1	0.060	60	
259		3		60	
262		1	0.016	16	SKEWED CRACK
262		2		16	
262		3		16	
265		1	0.020	20	
265		2		20	
265		3		20	
272		1	0.016	16	
272		2		16	
272		3		16	
273		1	0.005	5	
273		3		5	
278		1	0.020	20	
278		3		20	
282		2	0.013	13	
286		1	0.005	5	
286		3		5	
290		2	0.005	5	
295.5	CONST. JOINT	1	0.125	125	GROUTED, WATER LEAVING
295.5		2		125	AT FLOOR
295.5		3		125	
295.5		4		125	

STA	STA	LOCATION	WIDTH	WIDTH	REMARKS
302		1	0.005	5	
302		2		5	
302		3		5	
308		3	0.013	13	
310		1	0.005	5	
310		2		5	
316		1	0.005	5	
316		2		5	
316		3		5	
319		1	0.005	5	
319		2		5	
319		3		5	
324		1	0.030	30	
324		2		30	
324		3		30	
329		1	0.005	5	
329		2		5	
329		3		5	
331		2	0.005	5	
333		1	0.016	16	
333		3		16	
337		1	0.020	20	
337		2		20	
340		3	0.005	5	
340.5		3	0.005	5	
343		2	0.005	5	
346	CONST. JOINT	1		CM	WELL GROUTED
346		2		CM	
346		3		CM	
346		4		CM	
354.5		1	0.005	5	1" HOLE BOTTOM L. WALL
354.5		2		5	
354.5		3		5	
356		2	0.005	5	
363		2	0.005	5	
367.5		2	0.005	5	
370.5		3	0.005	5	
373		1	0.030	30	
373		2		30	
374.5		3	0.005	5	
375		3	0.005	5	
376.5		2	0.005	5	
384.5		1	0.005	5	
384.5		2		5	
384.5		3		5	

STA	STA	LOCATION	WIDTH	WIDTH	REMARKS
395.5	CONST. JOINT	1	0.125	125	GROUTED
395.5		2		125	
395.5		3		125	
395.5		4		125	
407		1	0.005	5	
407		2		5	
407		3		5	
412.5		1	0.005	5	
412.5		2		5	
412.5		3		5	
419		2	0.005	5	
421.5		1	0.013	13	
421.5		3		13	
434		1	0.005	5	
434		2		5	
438		1	0.005	5	
438		3		5	
445	CONST. JOINT	1	0.125	125	GROUTED
445		2		125	
445		3		125	
445		4		125	
457		2	0.005	5	
465		1	0.005	5	
465		3		5	
466		1	0.005	5	
466		2		5	
471		3	0.005	5	
474.5		1	0.005	5	
474.5		2		5	
474.5		3		5	
484		2	0.005	5	
484		3		5	
495	CONST. JOINT	1		CM	WELL GROUTED
495		2		CM	
495		3		CM	
495		4		CM	
501		3	0.005	5	
510		1	0.030	30	
510		2		30	
510		3		30	
519.5		1	0.020	20	
519.5		2		20	
519.5		3		20	
530		1	0.020	20	SKEWED CRACK
530		2		20	

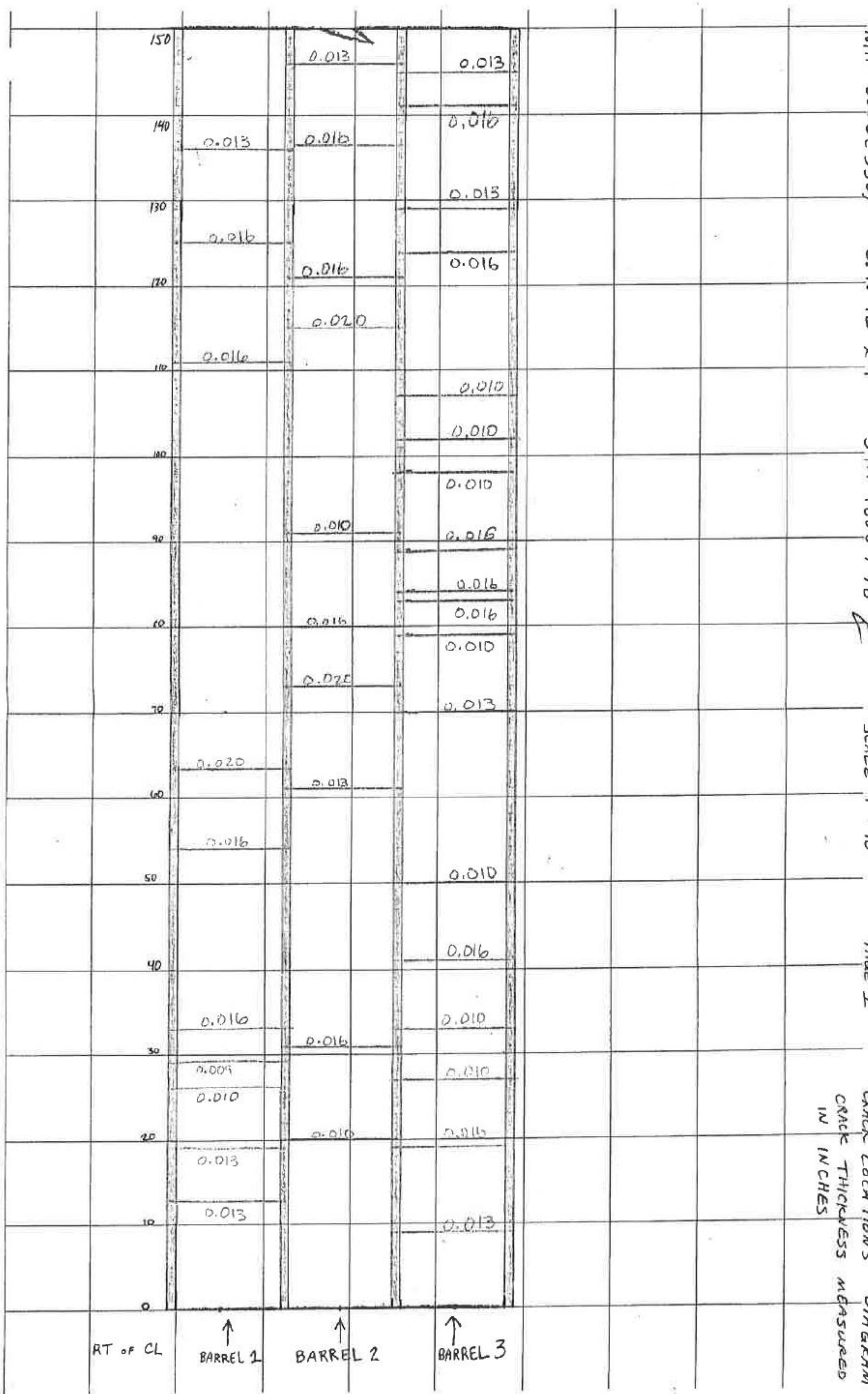
STA	STA	LOCATION	WIDTH	WIDTH	REMARKS
530		3		20	
535.5	5+37	1	0.013	13	SKEWED CRACK, HEAVY
535.5		2		13	SCALE IN CRACKS
535.5		3		13	
540		1	0.005	5	
540		3		5	
545	CONST. JOINT	1		CM	WELL GROUTED EXCEPT FOR
545		2		CM	WATER LOSS AT HOLE BOTTOM
545		3		CM	RIGHT WALL
545		4		CM	
556		1	0.005	5	
556		2		5	
556		3		5	
564		2	0.005	5	
564		3		5	
571.5		1	0.005	5	
571.5		3		5	
575		2	0.005	5	
581		2	0.005	5	
586		2	0.005	5	
587.5		3	0.005	5	
595		1	0.187	187	GROUTED, HOLES IN FLOOR
595		2		187	
595		3		187	
595		4		187	
608		1	0.005	5	
608		2		5	
608		3		5	
615.5		1	0.005	5	
615.5		3		5	
624		1	0.005	5	
624		2		5	
624		3		5	
633.5		3	0.005	5	
634		1	0.005	5	
634		2		5	
634		3		5	
645	CONST. JOINT	1	0.125	125	GROUTED, 1/8" CRACK IN FLOOR
645		2		125	
645		3		125	
645		4		125	
655		1	0.005	5	
659		1	0.005	5	
659		2		5	
663.5		1	0.005	5	

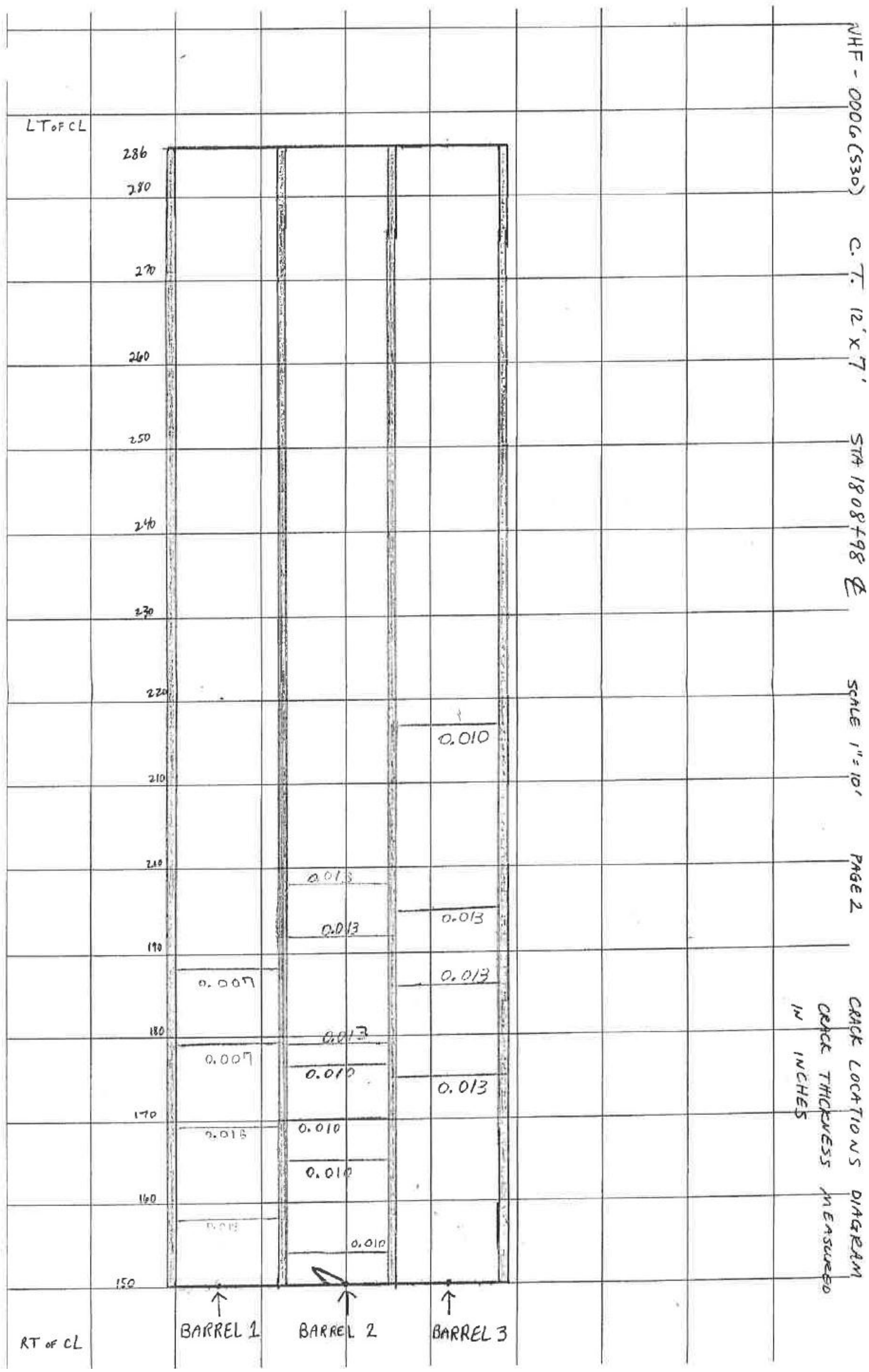
STA	STA	LOCATION	WIDTH	WIDTH	REMARKS
663.5		3		5	
664		1	0.005	5	
664		2		5	
681		1	0.005	5	
681		2		5	
681		3		5	
695	CONST. JOINT	1	0.187	187	GROUTED, WATER LEAVING
695		2		187	AT FLOOR
695		3		187	
695		4		187	
705.5		1	0.005	5	
705.5		2		5	
705.5		3		5	
710		1	0.005	5	
710		2		5	
720.5		1	0.020	20	
720.5		2		20	
720.5		3		20	
728		1	0.005	5	
728		2		5	
728		3		5	
738.5		1	0.005	5	
738.5		2		5	
738.5		3		5	
745	CONST. JOINT	1	0.125	125	GROUTED, WATER LEAVING
745		2		125	AT FLOOR
745		3		125	
745		4		125	
756		1	0.005	5	
756		2		5	
756		3		5	
768		1	0.005	5	
768		2		5	
768		3		5	
772.5		1	0.005	5	
772.5		3		5	
780		1	0.005	5	
780		3		5	
783		1	0.005	5	
783		2		5	
786.5		1	0.005	5	
786.5		3		5	
795	CONST. JOINT	1	0.125	125	GROUTED, 1/8" GAPS IN FLOOR
795		2		125	
795		3		125	

STA	STA	LOCATION	WIDTH	WIDTH	REMARKS
795		4		125	
805.5		2	0.005	5	
819.5		1	0.005	5	
819.5		2		5	
827.5		1	0.013	13	RUST SCALE PRESENT
827.5		2		13	
827.5		3		13	
838		1	0.005	5	
838		2		5	
838		3		5	
845	CONST. JOINT	1	0.125	125	GROUTED, WATER LEAKING IN
845		2		125	AT CEILING
845		3		125	
845		4		125	
854		1	0.016	16	
854		2		16	
854		3		16	
858		1	0.005	5	
858		2		5	
858		3		5	
866.5		1	0.04	40	
866.5		2		40	
866.5		3		40	
867.5		3	0.005	5	
873		1	0.02	20	
873		2		20	
873		3		20	
876		1	0.013	13	
876		2		13	
878		1	0.005	5	
878		3		5	
883.5		1	0.04	40	SKEWED CRACK
883.5		2		40	
883.5		3		40	
889		2	0.016	16	
895		1	0.005	5	
897		2	0.013	13	SKEWED CRACK

8+20	9+04	R. WALL CHAMFER
7+90	7+95	R. WALL CHAMFER
7+70	7+80	R. WALL CHAMFER
6+89	7+64	R. WALL CHAMFER
6+60	6+80	R. WALL CHAMFER
6+10	6+45	R. WALL CHAMFER
5+38	5+45	R. WALL CHAMFER
0+10	0+33	R. WALL CHAMFER

Centreville Culvert at 1808+98 ALDOT Condition Survey Data:




NHF - 0006(530) C.T. 12'x7' STA 1808+98 ℄
SCALE 1"=10'
PAGE 2
CRACK LOCATIONS DIAGRAM
CRACK THICKNESS MEASURED IN INCHES
LT OF CL
RT OF CL
286
280
270
260
250
240
230
220
210
200
190
180
170
160
150
0.010
0.013
0.013
0.013
0.007
0.013
0.007
0.010
0.013
0.010
0.010
0.010
BARREL 1
BARREL 2
BARREL 3

Appendix B

Temperature and Shrinkage Reinforcement Results

Appendix B contains additional data and example calculations from the temperature and shrinkage reinforcement analysis in Chapter 4 of this thesis. The ConcreteWorks temperature data used to calculate the thermal strain is shown in Figure B-1. Example calculation of the thermal shrinkage, drying shrinkage, creep coefficient, and joint movement used in the analysis is shown below. An example calculation for the analysis used is shown below as well. The example calculations were done in Microsoft Excel. The full results of the analysis are shown in Table B-1.

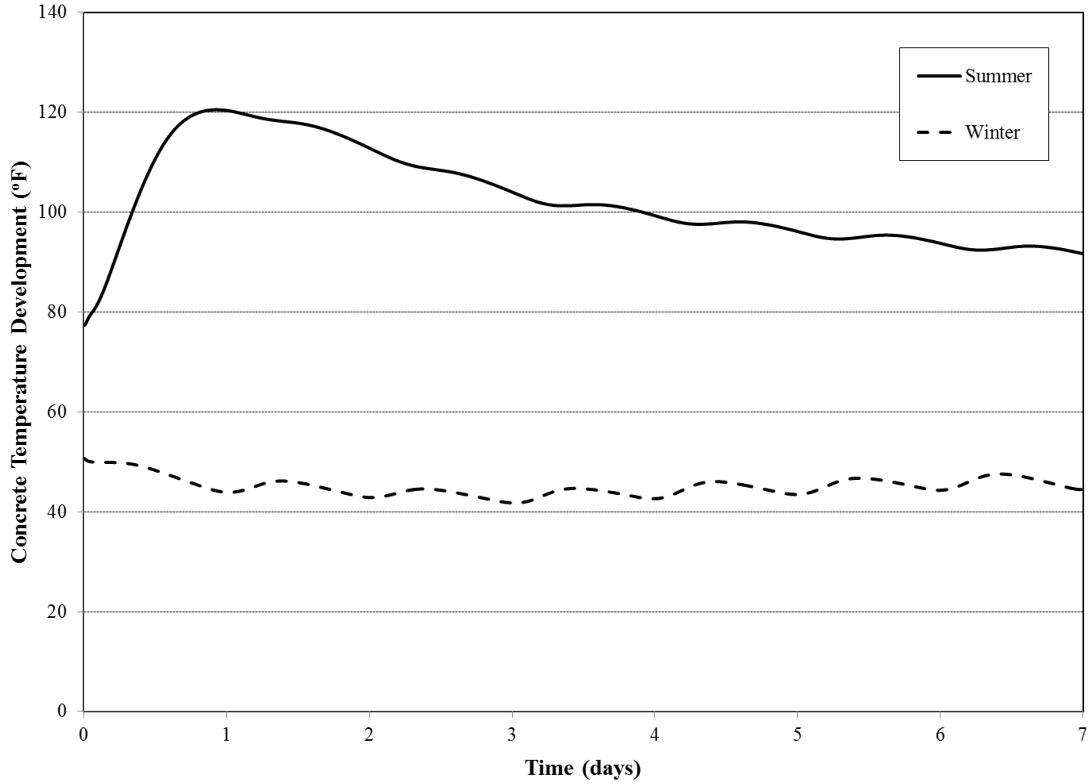


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Figure B-1: ConcreteWorks Temperature Data

Example Calculations:

Thermal Shrinkage

α_t = 6.00E-06 in./in./°F (Limestone Concrete)

T_{max} = 120.5 °F → $T_{zero} = 0.93 * T_{max}$ = 112.065 °F

T_{min} = 41.8 °F

$\Delta T = T_{zero} - T_{min}$

= 70.265 °F ($T_{zero} - T_{placement}$)

$\varepsilon_{therm} = \alpha_t * \Delta T$

= **-4.22E-04 in./in.** (Contraction)

Contraction Joint Movement

f'_c = 5,200 psi

E_c = 57000*SQRT(f'_c) = 4,110,328 psi

$\sigma^*_{c1} = f_t = 1.7f'^{2/3}_c$ = 510 psi

A_c = Concrete cross-section area above the base = 8,018 in.^{2}

W = Width of the base = 19.25 ft 231 in.

τ_f = Base friction = C_2 = 2.0 psi → Granular subbase (Rasmussen and Rizycki - Table 3)

F_{sh} = Shrinkage force = $\sigma_{av}A_c$

f_b = Distributed base friction Force = τ_f*W

Q = Fixed end force = Base restraint force = f_b*L/2

K = A_cE_c/L

$\Delta = K^{-1}(F_R - Q)$

Table B-1: Results for Joint Movement Calculations

Spacing	L (ft)	F_R (kips)	f_B (k/in.)	Q (k.)	K (k/in.)	**Δ (in.)**
3H	24	4,091	0.462	67	114,433	**0.035**
4H	32	4,091	0.462	89	85,825	**0.047**
36 ft	36	4,091	0.462	100	76,288	**0.052**
5H	40	4,091	0.462	111	68,660	**0.058**
6H	48	4,091	0.462	133	57,216	**0.069**
50 ft	50	4,091	0.462	139	54,928	**0.072**
7H	56	4,091	0.462	155	49,043	**0.080**

Drying and Autogenous Shrinkage (CEB MC90-99)

$\varepsilon_{cds}(t,t_c) = \varepsilon_{cdso}(f_{cm28})\beta_{RH}(h)\beta_{ds}(t-t_c) =$ $\boxed{\mathbf{-1.14E\text{-}04}}$ **Drying Shrinkage**

$\varepsilon_{cdso}(f_{cm28}) = [(220+110\alpha_{ds1})e^{\wedge}(-\alpha_{ds2}f_{cm28}/f_{cm0})]10^{-6} =$ 4.31E-04

$\alpha_{ds1} =$ 4 ACI 209.2R Table A.11

$\alpha_{ds2} =$ 0.12 ACI 209.2R Table A.11

$f_{cmo} =$ 1,450 psi Concrete Strength Constant

$f_{cm28} = f'_c + 1{,}160 =$ 5,160 psi

$f'_c =$ 4,000 psi

$\beta_{RH}(h) = -1.55[1-(h/h_o)^3]$ for $0.4 \leq h \leq 0.99\beta_{s1}$ -1.018

$h =$ 0.70 Avg RH

$h_o =$ 1 RH Constant

$\beta_{s1} = (3.5f_{cmo}/f_{cm28})^{0.1} \leq 1.0 \rightarrow$ 0.99834

$0.99\beta_{s1} =$ 0.988

$$\beta_{ds}(t-t_c) = \left[\frac{(t-t_c)/t_1}{350[(V/S)/(V/S)_o]^2 + (t-t_c)/t_1}\right]^{0.5} = \quad 0.260$$

$V/S =$ 7.5 in. Vol./Surface Ratio

$(V/S)_o =$ 2 in. Vol./Surface Constant

$t =$ 365 days Concrete Age

$t_c =$ 7 days Concrete Age at the Start of Drying

$(t-t_c) =$ 358 days Duration of Drying

$t_1 =$ 1 day Time Constant

$\varepsilon_{cas}(t) = \varepsilon_{caso}(f_{cm28})\beta_{as}(t) =$ $\boxed{\mathbf{-5.79E\text{-}05}}$ **Autogenous Shrinkage**

$\varepsilon_{caso}(f_{cm28}) = -\alpha_{as}[(f_{cm28}/f_{cmo})/(6+f_{cm28}/f_{cmo})]^{2.5}$ x10-6 = -5.92E-05

$\alpha_{as} =$ 700 ACI 209.2R Table A.11

$f_{cmo} =$ 1,450 psi Concrete Strength Constant

$f_{cm28} = f'_c + 1{,}160 =$ 5,160 psi

$f'_c =$ 4,000 psi

$\beta_{as}(t) = 1-\exp[-0.2(t/t_1)^{0.5}] =$ 0.978

$t =$ 365.00 days Concrete Age

$t_1 =$ 1 day Time Constant

$\varepsilon_{sh}(t,t_c) = \varepsilon_{cas}(t) + \varepsilon_{cds}(t,t_c) =$ $\boxed{\mathbf{-1.72E\text{-}04}}$ **Total Shrinkage**

Creep (CEB MC90-99)

$\varphi_{28}(t,t_o) = \varphi_o\beta_c(t-t_o) = \boxed{\mathbf{2.108}}$

$\varphi_o = \varphi_{RH}(h)\beta(f_{cm28})\beta(t_o) =$ 3.017

$$\varphi_{RH}(h) = \left[1+\frac{1-h/ho}{\sqrt[3]{0.1[(V/S)/(V/S)_o]}}\alpha_1\right]\alpha_2 = \quad 1.407$$

$h =$	0.70		Avg RH
$h_o =$	1		RH Constant
$V/S =$	7.5	in.	Vol/Surface Ratio
$(V/S)_o =$	2	in.	Vol./Surface Constant

$\alpha_1 = (3.5f_{cmo}/f_{cm28})^{0.7} =$ 0.988

$\alpha_2 = (3.5f_{cmo}/f_{cm28})^{0.2} =$ 0.997

$f_{cmo} =$	1,450	psi	Concrete Strength Constant
$f_{cm28} = f'_c + 1160$	5,160	psi	
$f'_c =$	4,000	psi	

$\beta(f_{cm28}) = 5.3/SQRT(f_{cm28}/f_{cmo}) =$ 2.810

$\beta(t_o) = 1/[0.1+(t_o/t_1)^{0.2}] =$ 0.764

$t_1 =$ 1 day

$t_o = t_{o,T}(\{9/[2+(t_{o,T}/t_{1,T})^{1.2}]\}+1)^{\alpha} =$ 2.590 days

$\alpha =$	0	Normal Concrete
$t_{o,T} =$	2.59	day
$t_{1,T} =$	1	day

$$\beta_c(t-t_o) = \left[\frac{(t-t_o)/t_1}{\beta_h+(t-t_o)/t_1}\right]^{0.3} = \quad 0.6987$$

$t =$	365 days	Concrete Age
$(t-t_o) =$	362 days	Duration of Loading

$\beta(h) = 150[1+1.2h/h_o)^{18}](V/S)/(V/S)_o+250\alpha_3 \leq 1500\alpha_3 \rightarrow$ 834.819

$\alpha_3 =$ 0.992

Temperature and Shrinkage Analysis

Input

$f_y =$ 60,000 psi

$f'_c =$ 5,200 psi

$f_t = 1.7(f'c)^{2/3} =$ 510 psi

$d_b =$ 0.5 in.

$L =$ 48 ft = 576 in.

$A_c =$ 1,440 in.^{2}

$A_s =$ 7.129 in.^{2} (Reinforcement at both faces)

$\rho = A_c/A_s =$ 0.0050

$E_c =$ 4,110,328 psi

$E_s =$ 29,000,000 psi

$n = E_s/E_c =$ 7.06

$\Delta =$ 0.06 in.

First Cracking

$s_o = d_b/10\rho =$ 10.1 in. = 0.84 ft

$N_{cr} = [3\Delta E_s(1+\rho n)-3Lf_tn+2s_of_tn]A_cA_s/[2s_onA_s-3LnA_s-2s_o(1+\rho n)A_c]$

$=$ 82,164 lbs

$\sigma_{s2} = N_{cr}/A_s =$ 11,525 psi

$\sigma_{s1} = (-2\sigma_{s2}s_o-3\Delta E_s)/(3L-2s_o) =$ -3,090 psi

$\sigma_{c1} = (N_{cr}/A_c)-\sigma_{s1}\rho =$ 72 psi

Final Cracking

$\varphi^* =$ 2.11 (Creep Coefficient) (ACI 209)

$E^*_e = E_c/(1+\varphi) =$ 1,322,367 psi

$\varepsilon^*_{sh} =$ -5.94E-04 in./in. (Therm. Shrink. + ACI 209 Drying Shrink.)

$\sigma_{av} = (\sigma_{c1} + f_t)/2 =$ 291 psi

$n^* = E_s/E^*_e =$ 22

$[4s_o^2f_tA_c]m^2 + [-6Lf_tA_cs_o+6\Delta E_sA_ss_o+6Ls_oA_sE_s(\sigma_{av}/E^*_e+\varepsilon^*_{sh})]m + [-9L\Delta E_sA_s-9L^2A_sE_s(\sigma_{av}/E^*_e+\varepsilon^*_{sh})]$

$0 =$ 3.00E+08 m^2 + -2.76E+10 m + 1.68E+11

$m =$ 85.55 , $\boxed{6.57}$

$m =$ 6.57

$s = L/m =$ <u>87.7</u> in. → 7.31 ft

$N(\infty) = [-3\Delta E_sA_s-A_sE_s(3L-2ms_o)(\sigma_{av}/E^*_e+\varepsilon^*_{sh})] =$ 657,564 lbs

$\sigma^*_{s2} = N(\infty)/A_s =$ 92,233 psi **<u>STEEL YIELDS!!</u>**

$\sigma^*_{s1} = (-2s_om\sigma^*_{s2}-3\Delta E_s)/(3L-2s_om) =$ -10,828 psi

$\sigma^*_{c1} = [N(\infty)-\sigma^*_{s1}A_s]/A_c =$ 510 psi

$w = \{-(\sigma^*_{c1}/E^*_e)[L-(2/3)ms_o]-\varepsilon^*_{sh}L-\Delta\}/m =$ **<u>0.012</u> <u>in.</u>**

Use if Steel Yields

$N(\infty) = A_s f_y =$ 427,761 lbs

$\sigma^*_{s1} = (n^*\rho f_{sy} + \varepsilon^*_{sh} E_s)/(1+n^*\rho) =$ -9,654 psi

$\sigma^*_{s2} = f_y =$ 60,000 psi

$\sigma^*_{c1} = (f_y A_s - \sigma^*_{s1} A_s)/A_c =$ 345 psi

$w = [-3\sigma^*_{s1} L - 2(f_y - \sigma^*_{s1}) - 3\Delta E_s]/3E_s =$ **0.12 in.**

Table B-2: Results for Temperature and Shrinkage Reinforcement Analysis

f'_c = 5,200 psi													
ε=-5.94E-4	φ^*=2.11	CULVERT J (w=0.012 in.)											
Joint Spacing	$A_{s,reqd}$ (in.^{2})	ρ	σ_{av}(psi)	σ^*_{c1}(psi)	σ^*_{s2}(psi)	σ^*_{s1}(psi)	σ_{c1}(psi)	σ_{s2}(psi)	σ_{s1}(psi)	Crack Spacing (ft)	Crack Width (in.)	No. Cracks	Δ (in.)
3H	6.470	0.0045	263	510	102,114	-11,443	16	0	-3,635	8.88	0.012	2.7	0.0352
4H	6.485	0.0045	263	510	101,865	-11,444	16	0	-3,591	8.82	0.012	3.6	0.0466
5H	6.852	0.0048	277	510	96,091	-11,146	43	5,821	-3,294	7.84	0.012	5.1	0.0523
6H	7.129	0.0050	291	510	92,234	-10,828	72	11,525	-3,090	7.31	0.012	6.6	0.0580
7H	7.067	0.0049	289	510	93,095	-10,882	67	10,617	-3,124	7.45	0.012	7.5	0.0692

Table B-3: Results for Figure 4-7

f_c = 5,200 psi													
ε=-5.94E-4	φ*=2.11	CULVERT J (w=0.012 in.)											
Joint Spacing	$A_{s,reqd}$ (in.^{2})	ρ	σ_{av}(psi)	σ^*_{c1}(psi)	σ^*_{s2}(psi)	σ^*_{s1}(psi)	σ_{c1}(psi)	σ_{s2}(psi)	σ_{s1}(psi)	Crack Spacing (ft)	Crack Width (in.)	No. Cracks	Δ (in.)
3H	9.840	0.0068	444	510	67,187	-7,481	378	54,321	-936	4.06	0.012	5.9	0.0000
3H	8.999	0.0062	391	510	73,001	-8,652	271	41,657	-1689	4.67	0.012	5.1	0.0088
3H	8.168	0.0057	341	510	80,211	-9,740	172	27,859	-2389	5.53	0.012	4.3	0.0176
3H	7.322	0.0051	296	510	89,620	-10,726	82	13,036	-3024	6.79	0.012	3.5	0.0264
3H	6.470	0.0045	263	510	102,114	-11,443	16	0	-3635	8.88	0.012	2.7	0.0352