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COLLEGE OF ENGINEERING

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# PLASTIC PIPE FOR HIGHWAY CONSTRUCTION PHASE 2

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**16. Abstract**

This report presents the results of a research project that investigated the use of thermoplastic pipes, namely high-density polyethylene (HDPE), polyvinyl chloride (PVC), and polypropylene (PP), as cross-drains under highways. It involved three distinct tasks: (1) evaluating plastic pipe performance under real-world conditions through field monitoring and case studies, which included evaluating the displacements of the pipes installed during the Phase 1 project; (2) establishing and demonstrating a practical product selection methodology that weighs the considerations between plastic, concrete, and metal pipes, and their various product forms; and (3) feasibility study and preliminary design of a test facility where installation conditions could be carefully controlled and load performance data accurately collected. The results from task (1) support the general conclusion from the Phase 1 project as well as studies by others that thermoplastic pipes have sufficient rigidity to be used for limited cross drain applications, but that the development of the necessary rigidity is highly dependent upon precisely and consistently following the soil type, bedding, compaction, and construction practices recommended by the manufacturers and governing industry organizations. The selection methodology task (2) resulted in checklists that can be easily used by highway engineers to consider all culvert pipe options equally and select the optimum material and class of pipe for cross-drainage application. The test facility task (3) resulted in the preliminary design of a testing apparatus that could simulate deep burial of pipes up to sixty inches in diameter, along with the accompanying instrumentation recommendations and test protocols.

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# ABSTRACT

This report presents the results of a research project that investigated the use of thermoplastic pipes, namely high-density polyethylene (HDPE), polyvinyl chloride (PVC), and polypropylene (PP), as cross-drains under highways. It follows a Phase 1 “Plastic Pipe for Highway Construction” project completed in 2011 that involved an analytical study into the allowable fill heights for thermoplastic pipes and a field study to observe the installation and performance of 1028 feet of HDPE and PVC pipe ranging from 36 to 54 inches in diameter in service conditions under varying fill depths and soil/backfill types. The present project involved three distinct components (1) evaluating plastic pipe performance under real-world conditions through field monitoring and case studies, which included evaluating the displacements of the pipes installed during the Phase 1 project; (2) establishing and demonstrating a practical product selection methodology that weighs the considerations between plastic, concrete, and metal pipes, and their various product forms; and (3) feasibility study and preliminary design of a test facility where installation conditions could be carefully controlled and load performance data accurately collected.

Four plastic pipe installations in Alabama were identified and inspected. However they have been in use for less than 15 years. The results of these investigations varied greatly, with some pipelines in acceptable condition while others were in poor condition. The test pipes constructed under the Phase 1 project (Beehive Road) were inspected manually and through robotic laser inspection, and revealed only minor isolated deformation and joint integrity concerns thus far. The results from this task support the general conclusion from the Phase 1 project as well as studies by others that thermoplastic pipes have sufficient rigidity to be used for limited cross drain applications (such as restrictions on ADT and burial depth), but that the development of the necessary rigidity is highly dependent upon precisely and consistently following the soil type, bedding, compaction, and construction practices recommended by the manufacturers and governing industry organizations. Recommendations from the case study and inspection tasks include continuing studies of these sites and others to develop more insight into the appropriate applications and appropriate limits for use of thermoplastic pipes.

The selection methodology task resulted in checklists that can be easily used by highway engineers to consider all culvert pipe options equally and select the optimum material and class of pipe for cross-drainage application. The accompanying condensed summary representing performance considerations extracted from the research serves as a guide. Crucial durability concerns include: sulfate content, resistivity, pH levels, abrasion, flammability, and ultraviolet radiation. Installation requirements include: minimum soil cover, maximum soil cover, culvert diameter, and presence of quality control personnel. Demonstrations of the approach are also presented.

The test facility tasks resulted in the preliminary design of a testing apparatus that could simulate deep burial of pipes up to sixty inches in diameter, along with the accompanying instrumentation recommendations and test protocols. The frame would provide the ability to test pipes in scenarios that simulate critical real-world variables such as fill height, soil and bedding type, foundation uniformity, compaction, pipe product, etc., at a time and cost efficiency far greater than that of in-situ installations and monitoring. The approach is also recommended for the validation of finite element models that could be used for broad parametric studies. In addition to rigidity and resistance tests, the apparatus could be used to evaluate joint integrity and hydraulic performance of pipes under loads.

**Keywords**: culvert, high density polyethylene, polyvinyl chloride, polypropylene, plastic pipe, cross-drain

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# ABBREVIATIONS

| | |
|---|---|
| AASHTO | American Association of State Highway and Transportation Officials |
| AC | Asphalt Concrete |
| ACI | American Concrete Institute |
| ACPA | American Concrete Pipe Association |
| ADOT | Arizona Department of Transportation |
| ADS | Advanced Drainage Systems, Incorporated |
| ADT | Average Daily Traffic |
| ALDOT | Alabama Department of Transportation |
| AOP | Aquatic Organism Passage |
| ASCS | AASHTO Soil Classification System |
| ASME | American Society of Mechanical Engineers |
| ASTM | American Society for Testing and Materials |
| BR | Burns-Richard |
| BLM | Bureau of Land Management |
| cfs | cubic feet per second |
| CALTRANS | California Department of Transportation |
| CDOT | Colorado Department of Transportation |
| ConnDOT | Connecticut Department of Transportation |
| DelDOT | Delaware Department of Transportation |
| DOT | Department of Transportation |

| | |
|---|---|
| EPA | Environmental Protection Agency |
| FDOT | Florida Department of Transportation |
| FHWA | Federal Highway Administration |
| FLH | Federal Lands Highway |
| FE | Finite Element |
| FEA | Finite Element Analysis |
| fps | feet per second |
| ft | feet |
| GAO | General Accounting Office |
| GDOT | Georgia Department of Transportation |
| HALS | Hindered Amine Light Stabilizer |
| HDPE | High Density Polyethylene |
| ID | Inside Diameter |
| IDOT | Illinois Department of Transportation |
| in | inch |
| KSDOT | Kansas Department of Transportation |
| ksi | kips per square inch |
| LaDOTD | Louisiana Department of Transportation and Development |
| lbs | pounds |
| LRFD | Load Resistance Factor Design |
| MaineDOT | Maine Department of Transportation |
| MDOT | Maryland Department of Transportation |
| MDOT | Mississippi Department of Transportation |

| | |
|---|---|
| mm | millimeter |
| MnDOT | Minnesota Department of Transportation |
| MoDOT | Missouri Department of Transportation |
| MAP-21 | Moving Ahead for Progress in the 21st Century Act |
| NBIS | National Bridge Inspection Standards |
| NCDOT | North Carolina Department of Transportation |
| NCHRP | National Cooperative Highway Research Program |
| NCSPA | National Corrugated Steel Pipe Association |
| NHDOT | New Hampshire Department of Transportation |
| NMDOT | New Mexico Department of Transportation |
| NYSDOT | New York Department of Transportation |
| ODOT | Oregon Department of Transportation |
| ORITE | Ohio Research Institute for Transportation and the Environment |
| PCA | Portland Cement Association |
| PCC | Portland Cement Concrete |
| pcf | pounds per cubic foot |
| PE | Polyethylene |
| PennDOT | Pennsylvania Department of Transportation |
| pH | Hydrogen Ion Concentration |
| pii | pounds per inch per inch of pipe |
| PPI | Plastics Pipe Institute |
| PPM | Parts Per Million |
| psi | pounds per square inch |

| | |
|---|---|
| PVC | Polyvinyl Chloride |
| RC | Relative Compaction |
| SCDOT | South Carolina Department of Transportation |
| SF | Safety Factor |
| USCS | Unified Soil Classification System |
| VDOT | Virginia Department of Transportation |
| WisDOT | Wisconsin Department of Transportation |
| WSDOT | Washington State Department of Transportation |
| $^{o}$C | degree Celsius |
| $^{o}$F | degree Fahrenheit |

# SECTION 1

# INTRODUCTION

## 1.1 Background

Culverts serve essentially the same function as bridges, but rarely receive the same level of attention. A culvert can be broadly described as a conduit that conveys water through a roadway embankment or past other flow obstructions. Culverts allow water to pass beneath roadways, protect against erosion and flooding, enhance the safety of pedestrian traffic, and allow the passage of farm animals. Culverts are especially useful for small streams or where building a bridge would be too expensive or impractical. An example of a double box culvert is shown in Figure 1-1.

Culverts exceeding a 20-foot span width are classified as bridges by the National Bridge Inspection Standards (NBIS) and must receive routine inspections in accordance with NBIS requirements. As stated in the FHWA *Hydraulic Design of Highway Culverts*, "Maintenance costs for culverts may result from channel erosion at the inlet and outlet, erosion and deterioration of the culvert invert, sedimentation, ice and debris building, and embankment repair in case of overtopping". (Schall et al. 2012)

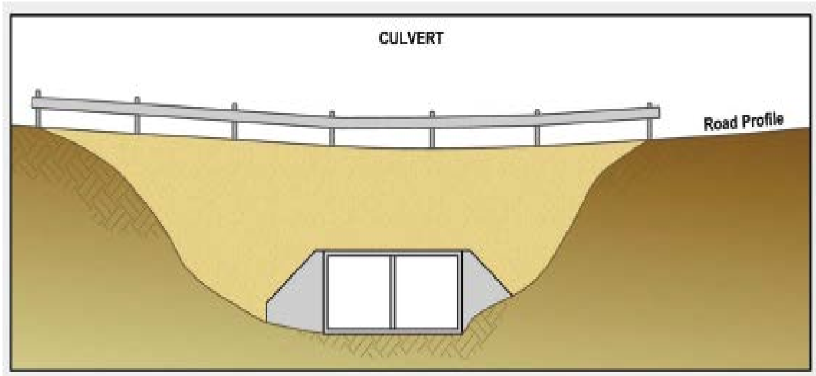



**Figure 1-1: Culvert Definition Example (Schall et al. 2012)**

The primary function of a cross-drain culvert is to convey the flow of surface water from the uphill side of a roadway to the other side. Culverts used for cross-drainage applications must support construction traffic, vehicle traffic, and earth loads. Therefore, culvert design must include both hydraulic and structural analyses. The ideal placement for a cross drain culvert is on a straight alignment with constant slope. Variations in alignment should only be used to accommodate unusual conditions as abrupt changes in direction or slope may negatively affect the hydraulic efficiency and lead to unforeseen maintenance issues. Figure 1-2 illustrates proper and improper culvert alignment.

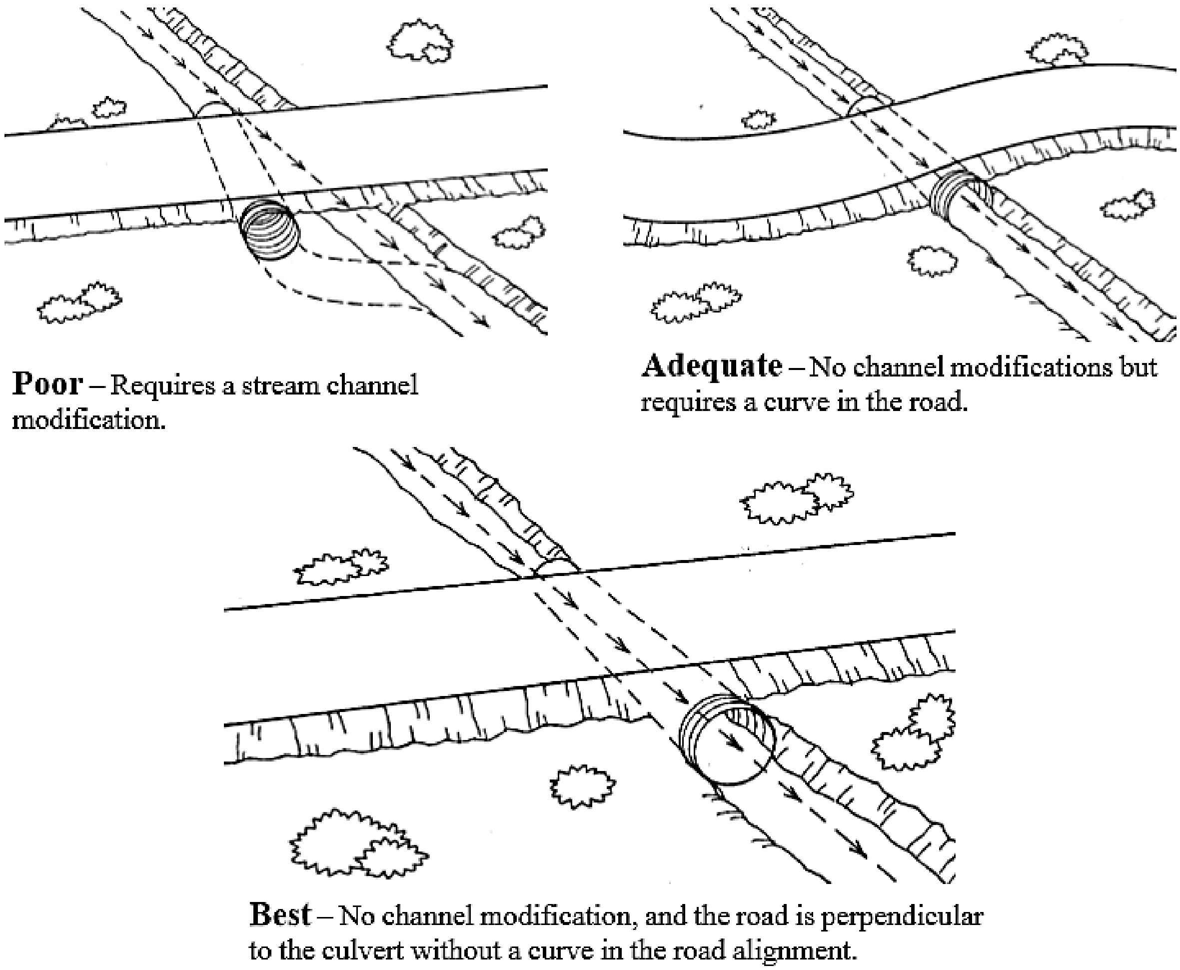



**Figure 1-2: Culvert Alignment beneath a Roadway (Keller 2003)**

According to the American Society for Civil Engineers Task Force on Hydraulics of Culverts, 13 specific design parameters are recommended as “Attributes of a Good Highway Culvert.” (CSPI 2007)

1. The culvert, appurtenant entrance and outlet structures should properly take care of water, bed load, and floating debris at all stages of flow.

2. It should cause no unnecessary or excessive property damage.

3. Normally, it should provide for transportation of material without detrimental change in flow pattern above and below the structure.

4. It should be designed so that future channel and highway improvement can be made without too much loss or difficulty.

5. It should be designed to function properly after fill has caused settlement.

6. It should not cause objectionable stagnant pools in which mosquitoes may breed.

7. It should be designed to accommodate increased runoff occasioned by anticipated land development.

8. It should be economical to build, hydraulically adequate to handle design discharge, structurally durable and easy to maintain.

9. It should be designed to avoid excessive ponding at the entrance which may cause property damage, accumulation of drift, culvert clogging, saturation of fills, or detrimental upstream deposits of debris.

10. Entrance structures should be designed to screen out material which will not pass through the culvert, reduce entrance losses to a minimum, make use of the velocity of approach in so far as practicable, and by use of transitions and increased slopes, as necessary, facilitate channel flow entering the culvert.

11. The design of the culvert outlet should be effective in re-establishing tolerable non-erosive channel flow within the right-of-way or within a reasonably short distance below the culvert.

12. The outlet should be designed to resist undermining and washout.

13. Energy dissipaters, if used, should be simple, easy to build, economical and reasonably self-cleaning during periods of easy flow.

Culvert materials are selected based upon the required structural strength, hydraulic roughness, durability, corrosion and abrasion resistance needed for a particular application. The three most common culvert materials being used through the early 2000's were concrete, corrugated aluminum, and corrugated steel (Normann et al. 2001). However, in 2006, the Federal Highway Administration (FHWA) published a regulation in the Federal Register that broadened the available types of culvert materials that could be specified and used for drainage applications on federal-aided highway projects. The new regulation required that “equal consideration” be given when specifying alternate pipe materials - including plastic and corrugated aluminum - that are “judged to be of satisfactory quality and equally acceptable on the basis of engineering and economic analysis” (Civil + Structural Engineer 2006).

In recent years, the use of thermoplastic profile wall pipes for culverts and other highway applications has increased (Gassman et al. 2005). The reason that thermoplastic pipes have been gaining popularity is largely because they are generally lighter, less costly per length, and more resistant to chemical attacks than most of the conventional pipe types. However, the rise in the use of thermoplastic pipes does not mean that the average civil engineer places a high level of confidence in plastic pipes. Thermoplastic pipes are still relatively new products for civil engineers, who tend to be much more familiar with steel and concrete (Sargand et al. 2002). As with any new material that has not been evaluated over its design life cycle, questions exist about the long-term structural performance of thermoplastic pipes. According to Sargand et al. (2002) the common questions or concerns about thermoplastic pipe are related to the allowable maximum fill height, the length of time required for stabilization of the pipe responses, and analysis and design methods. In order to quantify the long-term performance of the soil-pipe system, the performance of both the pipe itself and the interaction between the pipe and the soil envelope must be thoroughly investigated.

Although the most common types of plastic pipes, namely profiled wall high density polyethylene (HDPE) and polyvinyl chloride (PVC) pipes, and recently polypropylene, have been developed specifically for highway drainage applications and integrated into AASHTO standard specifications, there are still many concerns, and confidence in their use for cross-drain applications remains low. The most prominent of these concerns revolve around the long-term integrity of plastic pipes and their joints. Plastics such as HDPE and PVC are viscoelastic materials, and by definition, creep under loading, and their rigidity characteristics change considerably (Gabriel and Goddard 1999; Goddard 1994). Table 1-1 provides material stiffness and strength (modulus of elasticity and yield strength) for plastic pipes as defined by AASHTO and the Plastics Pipe Institute (AASHTO 2009, PPI 2003), along with comparable information for concrete and metal. The drastic stiffness change over time, which is used in standard design calculations, can be noted, with the modulus of polyethylene dropping by 80% and the modulus of polyvinyl chloride dropping by 65%; likewise, the yield stress used in design calculations for polyethylene and polyvinyl chloride drops by 70% and 47%, respectively. Furthermore, these properties are merely estimates based upon accelerated test methods and modeling that were adopted from the gas pressure pipe industry (PPI 2003, McGrath et al. 2009, Stuart et al. 2011). Although quality control of the raw material has improved over recent years, it has also been demonstrated that the resins used to manufacture plastic pipes can vary significantly between pipe producers. The long-term stability concern is further complicated by the fact that, unlike concrete pipes, plastic pipes are flexible-walled conduits whose strength and structural integrity relies upon the arching effect provided by the surrounding backfill. The arching effectiveness, along with tendencies for the soil and backfill to also creep through time, is highly dependent upon the backfill and compaction quality during installation.

**Table 1-1: Time-dependent properties of materials commonly used for drainage pipes (AASHTO 2009, PPI 2003)**

| Pipe Material Type | Initial | | 50-year | |
|---|---|---|---|---|
| | Yield (ksi) | Elastic Modulus (ksi) | Yield (ksi) | Elastic Modulus (ksi) |
| **Concrete (4 ksi)** | 4 (compression) | 3600 | No change | No change |
| **Aluminum (6061-T4)** | 30 | 10,000 | No change | No change |
| **Steel (50 ksi mild)** | 50 | 29,000 | No change | No change |
| **Corrugated PE pipe AASHTO M 294** | 3 | 110 | 0.90 | 22 |
| **Profile PVC pipe AASHTO M 304** | 7 | 400 | 3.70 | 140 |

## 1.2 Policy History

Appendix A to Subpart D of Part 635 of the *Code of Federal Regulations*, shown in Table 1-2, effectively excluded plastic pipe as an allowable culvert material on Federal-aided projects (GPO 2004). According to the Federal Highway Administration (FHWA), “When Appendix A was codified in 1974, the universe of available culvert materials was very limited and the state DOTs' experience with new culvert materials was equally limited” (Civil + Structural Engineer 2006). Subpart D General Material Requirements of Part 635 Construction and Maintenance sets forth conditions for the product and material selection on a Federal-aided highway project. As stated in the Subpart:

*Appendix A sets forth the FHWA requirements regarding (1) the specification of alternative types of culvert pipes, and (2) the number and types of such alternatives which must be set forth in the specifications for various types of drainage installations.* (GPO 2004)

**Table 1-2: Appendix A to Subpart D of Part 635 of the Code of Federal Regulations (GPO 2004)**

APPENDIX A TO SUBPART D OF PART 635—SUMMARY OF ACCEPTABLE CRITERIA FOR SPECIFYING TYPES OF CULVERT PIPES

| Type of drainage installation | Alternatives required | | | AASHTO designations to be included with alternatives | Application | Remarks |
|---|---|---|---|---|---|---|
| | Yes | No | Number | | | |
| Cross drains under high-type pavement.[1] | ............ | X ........ | .............. | ...................... | Statewide .......... | Any AASHTO-approved material.[2] |
| Other cross-drain installations. | X ........ | ............ | 3 minimum | M–170 and M–190. | ......do ............... | Do.[2] |
| Side-drain installations ....... | X ........ | ............ | ......do ....... | M–36 ............... | ......do ............... | Do.[2] |
| Special installation conditions. | ............ | X ........ | .............. | ...................... | Individual installation. | Specified to meet special conditions. |
| Special drainage systems (storm sewers, inverted siphons, etc.). | ............ | X ........ | .............. | ...................... | ......do ............... | Specified to meet site requirements. |

[1] High-type pavement is generally described as FHWA construction type codes I, J, K, L, and plant mix and penetration macadam segments, respectively shown in the right-hand columns of type codes G and H having a combined thickness of surface and base of 7 in or more (or equivalent) or that are constructed on rigid bases.
[2] Types not included in currently approved AASHTO specifications may be specified if recommended by the State with adequate justification and approved by FHWA.

The plastics industry lobbied congress, and in 2005, *The Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users* was signed into law by President George W. Bush. The Act guaranteed $244 billion in funding for highways, highway safety and public transportation, which made it the largest surface transportation investment in our Nation's history. Heavily favored by the plastics industry, the Act modified the former regulation and required equal consideration of alternative pipe material.

Section 5514 Competition for Specification of Alternative Types of Culvert Pipes now certified that "… *the Secretary shall ensure that States provide for competition with respect to the specification of alternative types of culvert pipes through requirements that* *are commensurate with competition requirements for other construction materials*" (Federal Register 2013). None of the 23 commenters, which included members of the American Concrete Pipe Association (ACPA) and the National Corrugated Steel Pipe Association (NCSPA), objected to the proposed changes after reviewing the *Notice of Proposed Rulemaking*. The FHWA offered the following comment in reaction to the signing of *The Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users*:

> *With the deletion of Appendix A, contracting agencies will no longer be able to cite Appendix A as their basis for not considering other culvert alternatives. The FHWA does not have a specific policy requiring the specification, number, and types of alternative materials for any other highway construction material. ... Thus, it is important to treat culvert materials the same as other materials by removing Appendix A.* (Civil + Structural Engineer 2006)

After expiring in 2009, the act was extended ten times until it was finally replaced by the *Moving Ahead for Progress in the 21st Century Act* (MAP-21), which was signed into law by President Barack Obama in 2012 and guaranteed more than $105 billion in funding for surface transportation programs. MAP-21 granted states the autonomy or sole authority to choose which culvert materials to use on a Federal-aided highway project. The significance of the word autonomy is defined by the FHWA as follows:

> *The use of the word "autonomy" in this section gives the State Departments of Transportation (State DOTs) and other direct recipients the sole authority and discretion to make a decision regarding culvert and storm sewer material types without any input or approval from the FHWA.* (FHWA 2012)

Although state departments of transportation were given the culvert selection autonomy through MAP-21, an increasing number of plastic pipes were now being chosen for transportation projects over concrete and galvanized steel pipes.

Thermoplastic pipe offers considerable advantages including a greater ease of transportation and handling and a greater resistance to corrosion and abrasion. However, many state transportation departments have little experience with thermoplastic pipe and are hesitant to revise conventional selection policies.

Concrete pipe and galvanized steel pipe are considered to be tried-and-true culvert materials. Numerous laboratory tests and field case studies have been performed. Furthermore, concrete pipes and galvanized steel pipes have successfully withstood the test of time. Concrete pipes have a longer expected service life and greater structural capacity compared to thermoplastic pipes, and therefore are still recommended for the majority of applications under routes classified as arterials and highways by state highway agencies. At county and municipal levels, small-diameter thermoplastic pipes are primarily used for drainage beneath driveways in suburban neighborhoods. (Gassman et al. 2005; Sargand et al. 2002)

### 1.3 Plastic Pipe for Highway Construction Project (Phase 1)

Due to the many concerns for using plastic pipes for cross drains, ALDOT sponsored the Auburn University Highway Research Center (AUHRC) to investigate the issues surrounding the use of plastic pipes for cross drain applications and to form recommendations for their use in Alabama through the effort entitled "Plastic Pipe for Highway Construction," which initiated in 2008 (henceforth referred to as the "Phase 1 project"). The Phase 1 project addressed its objectives through three distinct research components: (1) a comprehensive review of research literature, state-of-the-art practice, experiences and criteria of other state transportation agencies, and case studies; (2) finite element modeling to provide numerical data used for defining maximum and minimum fill heights; and (3) a field study involving 1028 feet of PVC and HDPE pipe of varying diameters and fill depths installed during construction of an interstate ramp near Auburn Alabama (henceforth referred to as the Beehive Road pipes). The Phase 1 project was completed in 2011 and culminated in a 350-page final report, Final Report Project 930-718, “Evaluation of HDPE and PVC Pipes Used for Cross-drains in Highway Construction,” that provides the details of all methodologies, findings and recommendations (Stuart et al. 2011).

However, through several subsequent interactions with ALDOT engineers, additional tasks and research needs were recommended, which can be summarized as the following three components:

**(1) Study of long-term plastic pipe performance should continue through monitoring the Beehive Road pipe installation**. The HDPE and PVC pipes installed as part of the Phase 1 project were set up for long-term monitoring. This includes vertical and horizontal diameter measurements taken at 10-feet increment stations that have been marked for continued monitoring. The condition of each pipe joint was also documented. It is therefore critical to monitor diameter changes, corrugation growth, and changes in joint integrity over time. Laser profiling was also conducted during the Phase 1 project, and it would be useful to get the comprehensive laser profiling data so that comparisons and assessments can be made through time.

**(2) Real-world construction effects should be further evaluated.** The intention of the Phase 1 Beehive Road study was to install the pipes by contractors under typical conditions and standard practices. Unfortunately, because of the notoriety of the project and involvement of plastic pipe producers in the planning phases of the Phase 1 project, it proved to be impossible to install the pipes without very close supervision by the manufactures' representatives. These representatives closely supervised every minute of installation of their respective pipes (PVC and HDPE), which would not be typical of real-world construction. The construction contractor also noted on several occasions that the installation, for example vibration compaction of #57 stone, was not typical practice. Over the past few years, ALDOT and county highway departments around the Alabama have installed and will continue to install plastic pipes under low and medium ADT roadways. Therefore, one readily available option for studying the performance of plastic pipe is to evaluate and monitor pipes that were part of ordinary county and state highway projects and therefore installed using standard practices.

Another option that has been discussed with DOT engineers would be to design and construct a test facility where installation conditions could be carefully controlled and load performance data accurately collected. Given the many potential combinations of product and construction parameters, along with the increasing likelihood that new culvert pipe products will emerge, a standardized testing resource and procedures would be a valuable investment.

**(3) Develop a "decision tree"**. The Phase 1 project thoroughly examined all of the factors and considerations for using plastic pipes for cross drain applications.

However, it did not distinctly weigh the advantages and disadvantages of plastic pipe against other culvert options for the purpose of making and defending pipe selection design decisions. The decision tree task would involve establishing a practical product selection methodology, or algorithm, that weighs the considerations between plastic, concrete, and metal pipes, and their various product forms, for a specific highway construction project and assists engineers with choosing the optimum solution.

Subsequent to the Phase 1 project, two other preliminary investigations were conducted as Auburn University Masters of Civil Engineering (MCE) projects, which were partially sponsored by the Auburn University Highway Research Center. The first was conducted by Doug Abernathy and resulted in the MCE project report entitled "Test Protocols for Evaluating the Structural Adequacy of Flexible Highway Drainage Pipes," which explored potential concepts for constructing an in-situ test apparatus that could be used to evaluate the load resistance performance of all types and diameters of pipes used for highway drainage purposes, subject to any combination of the burial depth loading, soil type, compaction, etc., within a controlled testing environment. The second MCE project was conducted by Chandler French, and resulted in the report entitled "Cross-Drain Pipe Material Selection Algorithm" that identified the primary constituents that should be involved in the selection approach.

## 1.4 Research Objectives

The overall objective of this study was to assist ALDOT with defining the issues, advantages and limitations for using plastic pipe for cross drain applications. Specific goals were to:

1. Evaluate the long-term performance of plastic pipes through continued monitoring of the HDPE and PVC pipes installed at the Beehive Road site.
2. Evaluate the implications of real-world construction procedures, with emphasis on the effects of backfill and compaction quality, through field studies of pipes already installed under roadways in Alabama using standard procedures.
3. Integrate all of the research findings from the Phase 1 "Plastic Pipe for Highway Construction" project and this "Phase 2" project into a practical selection methodology that can be used by state and county highway engineers to determine the optimum type and class of pipe (corrugated steel, aluminum, concrete, and plastic) for a specific project.
4. Investigate the potential benefits and costs of constructing a test facility that could be used to evaluate the load resistance performance of all types and diameters of pipes used for highway drainage purposes, subject to any combination of burial depth loading, soil type, compaction, etc., within a controlled testing environment.

## 1.5 Scope and Methodology

The project objectives were accomplished through the following tasks:

**TASK 1: Evaluate long-term performance of the HDPE and PVC pipes installed at the Beehive Road site**

This task involved returning to the Beehive Road installation to evaluate the changes that have occurred since the pipes were installed in 2011. The researchers revisited each of the pipes and measured the vertical and horizontal diameters at the 10 foot stations that were marked and measured during the Phase 1 project. The data was plotted against the original deflection data so that the changes are readily evaluated. Each of the pipes was also inspected for corrugation growth and joint integrity changes.

Prior to data collection, the research team prepared the pipes for inspection. The pipes have end entry protection that prevents research personnel from easily entering the pipes; furthermore some locations had significant vegetation and sediment buildup. The research team removed the end treatments when needed (and reinstalled once done with inspections) and removed the flow obstacles.

Also as part of Task 1, another laser scan was completed, similar to that done in 2011, but by a different company. This was intended to serve two broad purposes: (1) provide comprehensive diameter data and videography of each inch of each pipe that can be compared to the prior laser inspection and to future inspections, and (2) provide a demonstration that ALDOT can use to assess the potential for requiring a laser/video inspection of plastic pipes installed on highway projects, as other state highway agencies have done.

**TASK 2: Conduct field studies of existing pipes**

As suggested in a 14 January 2014 meeting with ALDOT engineers and management, it would be useful to compliment the Beehive Road data in which the installation was strictly overseen by pipe manufacturers' representatives with inspections of plastic pipes around Alabama that were installed under typical construction conditions. This task involved working with ALDOT and county engineers to identify candidate installations (already in place or installed during this proposed project), inspecting and collecting data on those installations, and analyzing and documenting the results.

**TASK 3: Develop a "decision tree" for pipe selection**

At the final project briefing of the Phase 1 project to ALDOT on 17 May 2011, it was commented that engineers need a methodology for deciding between metal, concrete, and plastic piping products for projects that involve cross drains (referred to as a "decision tree" during the discussion). The Phase 1 project thoroughly addressed all of the issues associated with PVC and HDPE pipes, but did not cover the comparative concerns associated with other types of culvert piping. Therefore, decision recommendations could not be adequately addressed in the Phase 1 plastic pipes project. As discussed briefly above, after the completion of the Phase 1 project, an AU MCE Student, Chandler French, completed an independent study entitled "Cross-Drain Pipe Material Selection Algorithm" that identified the primary constituents that should be involved in the selection approach; Chandler's work served as the starting point for Task 3 of the present Phase 2 project. This task therefore involved (a) defining the primary input parameters that should be involved in the "decision tree"; (b) interacting with

ALDOT and county engineers to ascertain the form and parameters of the decision tool that would be most useful to them; and (c) formalizing the methodology into a format that would be useful to highway engineers.

**TASK 4: In-situ pipe test design**

Since the flexible pipes used for drainage culverts rely on arching of the surrounding backfill for strength and stability, the pipes must be buried and backfilled following the trench conditions stipulated by the manufacturer in order to properly evaluate the structural performance of buried flexible pipe products. Parallel plate tests and other simple laboratory strength tests used for rigid pipes are not applicable. Furthermore, using field studies to examine structural performance of culvert pipe products have several shortcomings, including: (1) installation conditions cannot be controlled and/or may not be precisely known; (2) the data that can be collected is limited and collecting accurate data can be challenging (for example it is very difficult to measure and monitor soil pressure surrounding a cross drain culvert installed in the field); and (3) the design and construction parameters that can be explored (i.e., pipe diameters, base and backfill conditions, loading conditions, etc.) is very limited. Therefore, an alternative (or complimentary) approach would be to construct an in-situ test apparatus at the AU National Center for Asphalt Technology (NCAT) facility that could be used to evaluate the load resistance performance of all types and diameters of pipes used for highway drainage purposes, subject to any combination of the burial depth loading, soil type, compaction, etc., within a controlled testing environment. As discussed briefly above, an MCE study by Doug Abernathy entitled “Test Protocols for Evaluating the

Structural Adequacy of Flexible Highway Drainage Pipes” explored potential concepts. Task 4 of this Phase 2 project investigated the benefits and costs of constructing a test facility and expanded on the concepts investigated by Doug towards developing and proposing a preliminary design for constructing an in-situ test facility for buried pipe performance evaluation. The task therefore included: (a) researching test methodology and developing preliminary design concepts; (b) presenting preliminary design concepts to ALDOT engineers for feedback; (c) conducting design analyses; (d) conducting cost analyses; and (e) finalizing the preliminary design recommendation.

**TASK 5: Final reporting**

This comprehensive final report, which discusses all methodologies, data, analyses, and recommendations, was developed. A comprehensive briefing to the project advisory committee was presented on 19 April 2016.

The project was carried out by Auburn University faculty and students under the auspices of the Auburn University Highway Research Center. It was managed by Dr. Jim Davidson, Professor of Structural Engineering, as the principal investigator and assisted by AU graduate research assistants, Tim Deimler (Tasks 1 and 2), Megan Tocci (Task 3) and Todd Deason (Task 4).

### 1.6 Implementation and Benefits

It is critical that ALDOT make the best possible decisions regarding the use of plastic pipes for cross drain applications. The cost per linear foot of plastic pipe may be less than other competing products of the same diameter, but if the long term durability and issues related to installation quality are not sufficiently researched, the future costs of repairing and/or replacing plastic drainage pipes will be enormous. As discussed in detail in the Phase 1 “Plastic Pipe for Highway Construction” project report, a few states allow limited cross drain use of plastic pipes, but specific criteria in terms of maximum and minimum fill heights, installation requirements, ADT limitations, etc., vary greatly, indicating that there remains many concerns and unknowns for the use of plastic pipes under highways. This Phase 2 project played a vital step towards defining the advantages and limitations for using plastic pipes for cross drain applications so that ALDOT can develop and adopt safe and cost-optimized policies and procedures.

### 1.7 Report Organization

Sections 2, 3 and 4 represent the finding from Tasks 1 and 2 led by Tim Deimler. Section 2 presents a background review related to the construction and installation of plastic pipes that is important for the culvert inspection components of the research and reviews installation and long-term performance studies by others. Section 3 presents the findings of performance of plastic pipes that have been installed under Alabama roadways in recent years. Section 4 presents the deflection and performance data of the PVC and HDPE Beehive Road pipes that were installed in 2011, including the results of a laser deflection scan on all five pipelines. The full laser inspection report is provided in Appendix A.

Sections 5, 6 and 7 represent the finding from Task 3 led by Megan Tocci. Section 5 provides a thorough review of literature related to factors that must be considered in pipe material selection. The review is a compilation of information collected from many sources including state departments of transportation, the Transportation Research Board (TRB), the University of South Carolina, the Plastics Pipe Institute (PPI), the National Corrugated Steel Pipe Association (NCSPA), the American Concrete Pipe Association (ACPA), the American Society for Testing and Materials (ASTM), and the American Association of State Highway and Transportation Officials (AASHTO). The literature review focused on detailed evaluation of each culvert material type based on durability concerns, service life expectations, and installation requirements. A tabulated comparison of each culvert material concludes the section. Section 6 provides the decision process and pertinent information that was used to create *The Specification for Culvert Material Selection.* Section 7 provides several demonstrations of the effectiveness of *The Specification for Culvert Material Selection*. The demonstrations represent culvert installation projects throughout the United States. Appendix B provides *The Specification for Culvert Material Selection*. Appendix C analyzes culvert material selection based exclusively on average daily traffic flow counts. Appendix D presents a culvert material selection guide developed by Kevin White of E. L. Robinson Engineering entitled *Alternate Pipe Material Selection Protocol*.

Sections 8 through 12 represent the findings from Task 4 led by Todd Deason. Section 8 is a literature review that introduces other testing apparatuses, previous plastic pipes experiments by others, and existing thermoplastic pipe testing setups. Section 9 presents testing options and the possible locations of installing a test facility. Section 10 presents the preliminary design of a test frame. Section 11 describes potential testing capabilities of the testing apparatus. Section 12 defines protocols on some of the tests that could be conducted with the apparatus. Appendix E presents a summary of flexible pipe testing by Jeyapalan and Watkins (2004); Appendix F presents the design calculations and other details of the test apparatus; and Appendix G presents a cost analysis of the test apparatus.

Section 13 provides a broad summary, conclusions and recommendations resulting from the project.

# SECTION 2

# FIELD MONITORING BACKGROUND

## 2.1 Construction and Installation

In order to attain long term life of buried pipe, contractors must be knowledgeable of the basics of the pipe/soil composite structures so that they will be able to anticipate or avoid any problems or damage that will arise with poor installation or construction practices. This subsection summarizes the procedures and guidelines for proper installation of plastic pipe as given by the Plastic Pipe Institute (PPI 2008). The subsequent subsections highlight specific projects that were conducted to evaluate the performance of plastic pipes for cross drain applications. Additional case studies and research background are also provided in Section 5.

### 2.1.1 Importance of Installation and Construction

The overall performance of thermoplastic pipes is highly dependent on proper installation. Every action performed by the contractor, transporter, and yard handler will affect the performance and durability of the pipeline. Using correct procedures for the trench excavation, pipe laying and pipe joining are essential since the structural design of a buried pipeline presumes the response to loads of a pipe/soil composite structure. The correct selection and compaction of soils composing the pipe/soil envelope are likely to strongly influence the structural performance of both pipe and soil. Without proper installation, the desired surrounding constant pressure that supports the pipe will not be achieved. Figure 2-1 illustrates the terminology commonly associated with culvert pipe construction and used in this report.

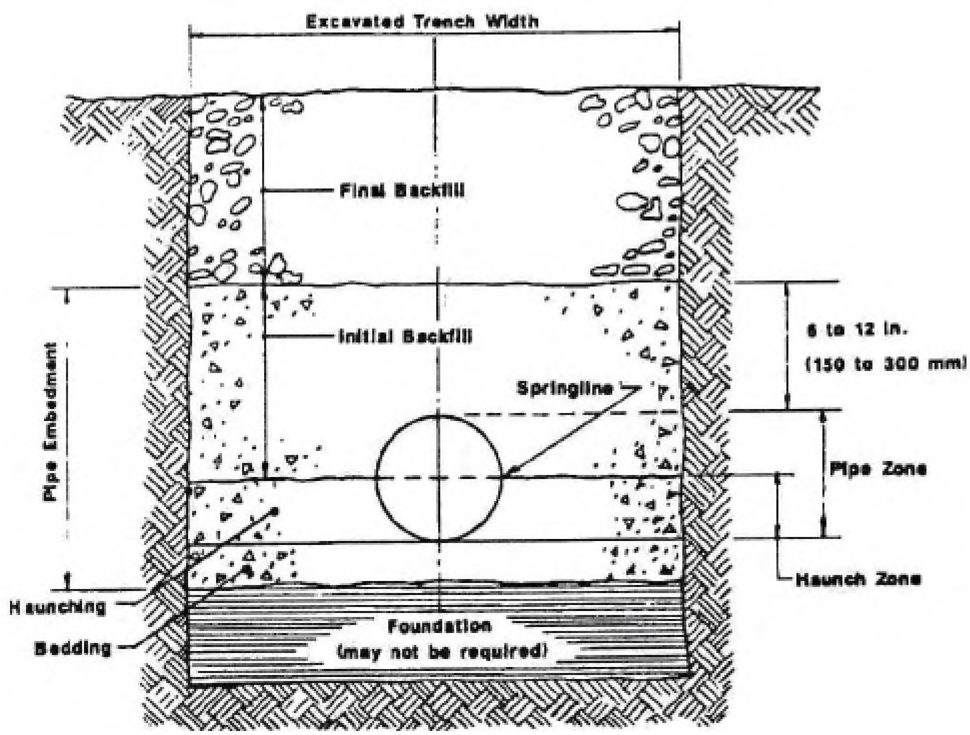



**Figure 2-1: Pipeline Cross Section with Common Terminology (ASTM D2321)**

2.1.2 Pipe Transporting and Handling

When the piping materials arrive at the construction site, the contractor should thoroughly inspect each pipe segment, including the pipe walls and corrugations, gaskets, pipe ends, couplers or other joints and accessories for damage such as cuts, gouges, delamination, bulges, flat areas, and ovality that may have occurred during transporting. When unloading and handling pipe segments, manufacturers' instructions for properly unloading the product off of the truck or carrier must be followed. Pipe segments should not be dropped or rolled at any time of the transportation or handling. For pipes that are 18 inches in diameter or less, the pipe may be hand lifted and placed by two people. For larger diameters, mechanical equipment is required using a minimum of two lifting slings placed at third points about the length of the pipe. Slings should be made of cloth; metal chains or cables should not be used. Loading booms and forklifts should be avoided because they can cause damage to the pipes.

When storing pipe on the jobsite, palletized pipe should remain on pallets. Non-palletized pipe should be stockpiled in an area free of construction traffic on flat ground free of debris. Tie-down straps or bands should remain until the pipe is properly secured. Stockpiles should be on level ground and should be a maximum of 6 feet high. Each pipe segment should be supported in a way to avoid concentrated forces along bell ends. Exposed gaskets should be protected from dust and sunlight exposure. In some cases, extreme summer heat can affect the ovality of some pipes. To address this, pipe segments should be rotated during storage.

### 2.1.3 Trench Preparation and Excavation

ASTM D2321 should be followed when preparing a trench for the installation of a pipeline. This provision states that the trench width should be no wider than required to safely and conveniently compact backfill material on either side of the pipe. The width of the trench is dependent on the backfill material, ease of compacting the backfill in the haunch zone, compaction methods, pipe diameters and the width of the nearest larger size excavator bucket (PPI 2008). AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing* Section 30 requires a minimum trench width of not less than 1.5 times the pipe outside diameter plus 12 inches (AASHTO 2008). ASTM D2321 states that trench widths should be the greater of either the pipe outside diameter plus 16 inches or 1.25 times the pipe outside diameter plus 12 inches. Table 2-1 defines the minimum trench width for diameters ranging from 4 to 72 inches.

**Table 2-1: Minimum Trench Width (PPI 2008)**

| Inside diameter in. (mm) | Typical Outside Diameter in. (mm) | AASHTO Sec 30 Min. Trench Width in. (mm) | ASTM D 2321 Min. Trench Width in.(mm) |
|---|---|---|---|
| 4 (100) | 5 (120) | 19 (480) | 21 (530) |
| 6 (150) | 7 (177) | 22 (570) | 23 (580) |
| 8 (200) | 9 (233) | 26 (650) | 25 (640) |
| 10 (250) | 11 (287) | 29 (740) | 27 (690) |
| 12 (300) | 14 (356) | 33 (840) | 30 (760) |
| 15 (375) | 18 (450) | 39 (980) | 34 (870) |
| 18 (450) | 21 (536) | 44 (1110) | 38 (970) |
| 21 (525) | 24 (622) | 49 (1240) | 43 (1080) |
| 24 (600) | 27 (699) | 53 (1350) | 46 (1180) |
| 30 (750) | 34 (866) | 63 (1600) | 55 (1390) |
| 36 (900) | 41 (1041) | 73 (1870) | 63 (1610) |
| 42 (1050) | 48 (1219) | 84 (2130) | 72 (1830) |
| 48 (1200) | 54 (1372) | 93 (2360) | 80 (2020) |
| 54 (1350) | 61 (1577) | 105 (2670) | 90 (2276) |
| 60 (1500) | 67 (1707) | 113 (2870) | 96 (2440) |
| 72 (1800) | 80 (2032) | 132 (3350) | 112 (2840) |

If two pipes are placed parallel within the same trench, soil must be properly compacted in the region between the two pipes. The minimum spacing requirements between the two pipes are shown in Table 2-2. These dimensions may need to be increased depending on the type of backfill, the compaction equipment and joining methods.

**Table 2-2: Minimum Spacing of Parallel Pipes (PPI 2008)**

**Minimum Spacing of Parallel Pipes in a Single Trench**

| Normal Diameter (D) in. (mm) | Minimum Spacing in. (mm) |
|---|---|
| ≤ 24 (600) | 12 (300) |
| > 24 (600) | D/2 |

Dry trench conditions are a prerequisite for proper placement and embedment of drainage pipe (PPI 2008). Surface water should be drained away from the trench. Water in the trench can cause the pipe to float and interfere with the maintenance of line and grade. The water will also compromise trench walls and slopes. Ground water should not rise above the trench bottom until after the installed pipe is fully bedded and enough fill has been placed to prevent floatation. Sheeting or sharing placed in the site should not allow seepage of water and soil in areas where groundwater is higher than the trench bottom. Seepage can cause the soil fines to be lost, which can create soil voids in the vicinity of the pipe. For ideal installation, soil trenches should be dry without the presence of any surface or ground water.

### 2.1.4 Placing and Connecting Pipes

When lowering pipe sections into a trench, workers should be careful not to damage the pipe or pipe ends at the couplings. When the pipeline assembling is halted for a period of time, the pipe ends should be covered to prevent dirt, water, or animals from entering the line. Many problems could arise when soil is able to enter the pipe joints, and therefore joints are designed to be soil tight.   Soil lost through joints will change the support of the pipe at the springline and invert, and the structural integrity of the pipe/soil interaction will be compromised. This loss in dirt will also cause voids between pipe and the ground above resulting in road or other structure collapse into the void.

Different joint types are available. Split couplings are used as soil tight joints, while gasket joints are used when water tightness is needed. Contractors should follow manufacturers' instructions during installation. All joints should remain free of mud or grit to ensure effective joining and sealing. Pipe should be laid so that the bells are pointed upstream. Pipe manufacturers can supply materials needed to aid in pipe installation such as tees, wyes, elbows, end caps, and other style fittings.

Manholes are commonly installed along culvert pipelines to facilitate changes in pipe size, grade or direction, and cleanout access. Precast concrete manholes and catch basins are manufactured with openings for the line. Both the inlet and outlet connection holes in the manhole are slightly oversized in order to receive a standard size pipe. Grouting is used to secure and seal the remaining void space from the outside of the pipe and the manhole openings. Rubber "boots" are also available depending on the needs of the system being built.

### 2.1.5 Bedding, Haunching and Backfill

Before bedding is established, the trench bottom should be created using stable soils that are clear of protruding rocks. This may require over-excavation of the native soil before it is replaced with a material that is a suitably-graded soil mixture. This will impede the migration of fines and subsequent loss of pipe support. The role of bedding is to establish the line and grade for the pipeline. Bedding also provides a firm, but not rigid, pipe support. Bedding material is usually a compacted granular material that should be spread evenly and compacted uniformly over the flat trench foundation. Approximately four inches of bedding is placed and compacted on the foundation so that load distributions are equal along the invert of the pipe. The pipe is then set onto the bedding and backfilled under the haunches.

The haunch zone provides the majority of the resistance against soil and traffic loadings. The material used for backfill should be placed uniformly on each side of the pipe. Larger layers or lifts can usually be used when placing more angular backfill materials. Smaller and rounder soil particles are usually placed in smaller lifts. The backfill for this zone should be shoveled under the pipe. All voids should be filled before compaction, if needed, is performed. Compaction should not disturb or change the pipes' alignment or angle. Backfill should be continuously added until it is up to the springline of the pipe, which is the end of the haunch zone.

Initial backfill should be placed a minimum of six inches above the crown of the pipe. Its role is to provide pipe support and pipe protection from stones in the final backfill. For HDPE pipe, Class I, II, III and low plasticity Class IVA materials may be used. Use of the Class IVA fine-grained inorganic low to medium plasticity materials are discouraged. Compaction for this material must take place at or near optimum moisture levels to achieve the required density. This material may not be suitable under high fill heights or surface wheel loads. High plasticity clays and silts are also not recommended for the initial backfill.

### 2.1.6 Compaction

The quality of compacted fill in the embedment zone must be high in order to get the proper performance of the flexible pipe. The chance of soil loads and wheel loads to be attracted away from the pipe by soil adjacent to the pipe is increased with denser fill soils. The denser soils also tend to lessen the potential of pipe ovality. Proper bedding in the embedment zone allows the flexible pipe to perform in a stable and more predictable manner. Proper compaction will lead to better structural support and can prevent deformation.

All embedment materials should be worked to insure uniform compaction. Hand-held mechanical tampers should be used between the pipe and trench walls. Vibratory equipment is preferred for the clean coarse-grained crushed stone, gravels, and sand of Classes I and II. Jumping jacks and walk behind vibratory rollers are used to provide the vibration and impact force needed for fine soils with high plasticity. Care should be taken during the placement and compaction of the embedment side fill. Workers should ensure that they do not elongate the vertical diameter past the limits of the manufacturer’s recommendations. Direct contact between the pipe and any compaction equipment should be avoided. Before any heavy compaction or construction equipment travels over the pipeline, sufficient backfill should be placed to prevent damage. ASTM D2321 requires at least six inches of backfill above the crown of the pipe. Under structures such as roads, there should be at least twelve inches of cover over the pipe before using any compaction equipment. Three feet of cover is required before using any ride-on equipment.

## 2.2 Ohio's Long-term Monitoring of Pipes under Deep Cover

The following section summarizes the long-term monitoring of plastic pipes under deep cover performed by the Ohio Research Institute for Transportation and the Environment that was led by Sargand and Masada (2007).

### 2.2.1 Background

The Ohio Research Institute for Transportation and the Environment (ORITE) began a comprehensive field study near Athens, Ohio in the summer of 1999 to evaluate the performance of large diameter thermoplastic pipes under deep soil cover. The research was conducted to help obtain additional information about thermoplastic pipe performance, specifically pipes greater than twenty-four inches.

### 2.2.2 Construction

In the summer of 1999, construction began on the deep burial project. Eighteen thermoplastic pipes were installed. Six of these were polyvinyl chloride (PVC) pipes and twelve were made from high density polyethylene (HDPE). Each pipe was instrumented with linear potentiometers, strain gauges, earth pressure cells, and other sensors under either 20 or 40 feet of soil cover. Figure 2-2 and Table 2-3 depict the characteristics and installation conditions for each pipe.

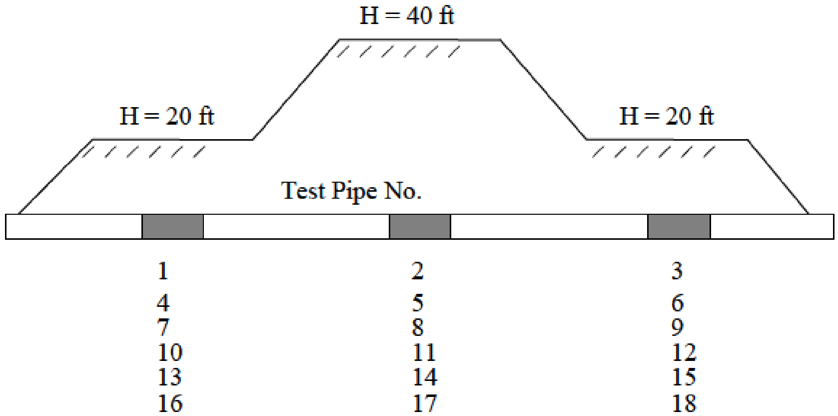



**Figure 2-2: Fill Heights over Thermoplastic Pipes (Sargand and Masada 2007)**

**Table 2-3: Pipe Installation Characteristics (Sargand and Masada 2007)**

| Pipe No. | Pipe Material | Nom. ID (inch) | Wall Type | Backfill: Type | Backfill: RC | Final Fill Height (ft) | Bedding Thickness (inch) |
|---|---|---|---|---|---|---|---|
| 1 | PVC | 30.0 (762 mm) | A | Sand | 96 | 20 (6.1 m) | 6 (150 mm) |
| 2 | | | | Crushed Rock | 96 | 40 (12.2 m) | |
| 3 | | | | | 86 | 20 (6.1 m) | |
| 4 | | | B | Sand | 86 | 20 (6.1 m) | |
| 5 | | | | Crushed Rock | 96 | 40 (12.2 m) | |
| 6 | | | | | 96 | 20 (6.1 m) | |
| 7 | HDPE | 30.0 (762 mm) | C | Sand | 96 | 20 (6.1 m) | |
| 8 | | | | | 96 | 40 (12.2 m) | |
| 9 | | | | C. Rock | 86 | 20 (6.1 m) | |
| 10 | | | D | Sand | 86 | 20 (6.1 m) | |
| 11 | | | | Crushed Rock | 96 | 40 (12.2 m) | |
| 12 | | | | | 96 | 20 (6.1 m) | |
| 13 | HDPE | 42.0 (1,067 mm) | E | Sand | 90 | 20 (6.1 m) | 0-12 (0-300 mm) |
| 14 | | | | | 96 | 40 (12.2 m) | 3.2-15.2(80-380 mm) |
| 15 | | | | Crushed Rock | 90 | 20 (6.1 m) | 0-12 (0-300 mm) |
| 16 | | 60.0 (1,524 mm) | F | | 90 | 20 (6.1 m) | 3.2-9.2 (80-230 mm) |
| 17 | | | | | 96 | 40 (12.2 m) | 6-12 (150-300 mm) |
| 18 | | | | Sand | 96 | 20 (6.1 m) | 3.2-9.2 (80-230 mm) |

[Note] ID = Inside Diameter; and RC = Relative Compaction.

Six different profile-wall types were used in the eighteen pipes installed in the test site. The profile-wall types include Type A (hollow beam PVC by Vylon), Type B (corrugated PVC by Contech), Type C (corrugated HDPE by Lane), Type D (corrugated HDPE by ADS), Type E (corrugated HDPE by ADS), and Type F (honeycomb HDPE by ADS). Specific Dimensions of these profile-wall dimensions are illustrated in Figure 2-3.

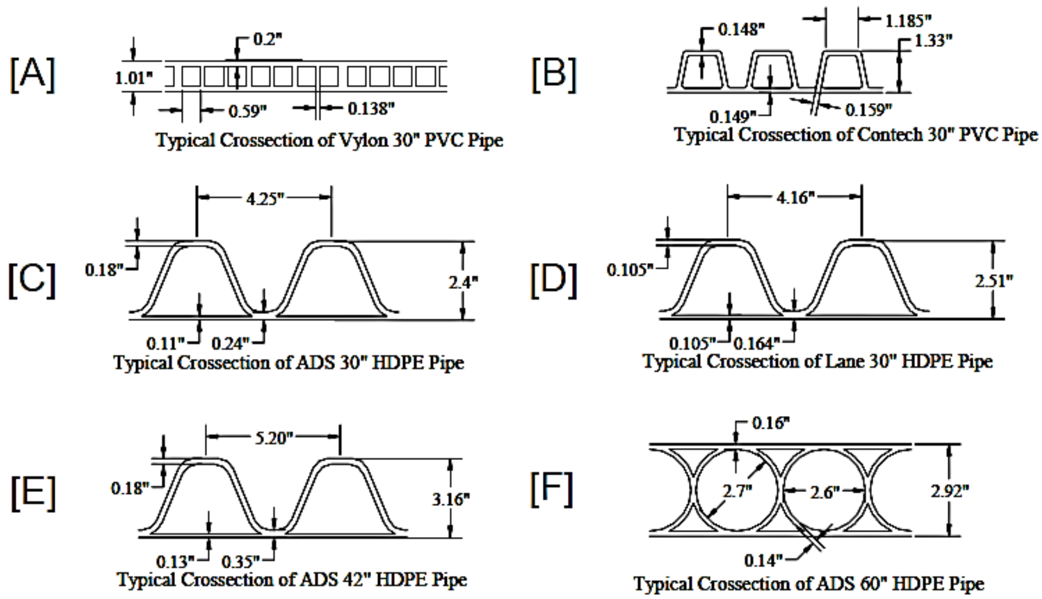



**Figure 2-3: Dimensions of Profile-Wall Designs (Sargand and Masada 2007)**

2.2.3 Pipe Deformations and Performance

After initial backfill was placed, a vertical deflection of 0.3% to 0.8% and horizontal deflections of 0.2% to 0.6% were measured in the test pipes. During the first year, every pipe performed satisfactorily under the deep soil cover. The pipes had small deflection changes; no signs of structural distress or failure were found on the interior surface of any pipes. The vertical deflection experienced after this first year was less than -4.0% (negative indicates decrease in diameter), while the horizontal deflection measured was less than 2.0% (increase). Circumferential shortening was also low with less than -1.0%. Vertical soil pressure measured at the crown ranged from 6 to 14 psi under 20 feet of fill and from 11 to 25 psi under 40 feet of soil fill. Lateral soil pressure measured at the springline ranged from 6 to 17 psi under 20 feet of fill and from 9 to 25 psi under 40 feet of fill.

Researchers observed that the dry density of the compaction initially achieved on the backfill of the soil had a major influence on both the short and long-term deflections. Further observations were based on the deflection lag factor, which was calculated by dividing the end-of-construction vertical deflection by the long-term vertical deflection. This deflection lag factor remained between 1.0 and 1.4 and was slightly larger in the crushed rock backfill than in the sand backfill. Pipes in the sand backfill had a larger deflection ratio than in the crushed rock backfill. The denser the backfill soil, the larger the deflection ratio for the pipes. Circumferential shortening in the sand backfill was larger than in crushed rock backfill.

Differing trends were also observed between the HDPE or PVC piping materials. Under very similar installation conditions, HDPE pipes deflected vertically more than the PVC pipes. However, these same HDPE pipes had less horizontal deflection than the PVC pipes. The overall circumference shortened more in the HDPE than the PVC. HDPE pipes deflection lag factor was slightly smaller than the PVC.

Soil pressures around each thermoplastic pipe declined slightly each year and then underwent seasonal cyclic fluctuations while pipe deflections were relatively constant. These seasonal fluctuations are believed to be caused by pipe material temperatures, soil moisture, and other environmental factors. Saragand and Masada state that not only the pipe material and size played a role in determining the magnitude of thermally induced soil pressure around pipe, but also the pipe’s exterior geometry in relationship to gradation characteristics of backfill soil. Seasonal soil pressure was more pronounced in PVC pipes under 20 feet compared to HDPE pipes at the same depth. This difference was observed even though the coefficient of thermal expansion was twice as high in HDPE than in PVC.

Using the deflection data measured over the five year project in each thermoplastic pipe, researchers then attempted to evaluate the best mathematical function to describe the time-dependent nature of these deflection changes. The final equation chosen was a logarithmic function that provided predictions for pipes' future dimensions and overall deflection. In a 100 year period, a vertical deflection of less than -5.0% is expected to be measured in the majority of the pipes. The other pipes were predicted to experience a maximum deflection of -7.5%.

The integrity of pipe joints was evaluated by the ORITE team upon visual inspection of the interior of selected pipes. Joints in each pipeline performed satisfactorily. Researchers did not detect any openings, offsets, or measurable movements. No structural problems such as cracks or buckling were found. Joints were sealed properly, which prevented backfill infiltration.

### 2.2.4 Ohio Long-term Monitoring Study Summary

ORITE's long-term monitoring project of pipes under deep cover showed that proper initial installation is the key to assuring stable long-term structural performance of thermoplastic pipes. Soil pressure acting against the pipe and the pipe deflections undergo seasonal fluctuations each year. These cyclic fluctuations are insignificant compared to the amount of soil pressure and deflections that are formed by gravitational loading from embankment fill. With proper installation, the pipe interacts with the surrounding soil so that its performance is not significantly affected by creep, wall buckling, or cracking for at least five years. The quality of the construction and installation is the determining factor for both the short and long-term performance of flexible thermoplastic piping products.

To ensure that these projects perform adequately over long periods of time, ORITE researchers provided three general installation parameters. These parameters are for pipe products installed with 8-inch thick lifts of 96% compaction granular backfill within a properly prepared trench, which allows the pipes' structural integrity to stabilize shortly after construction. Thermoplastic pipes must be installed:

- In a trench having stable trench walls and a minimum trench width of two times the pipe outer diameter (OD) or the pipe OD plus 4 feet, whichever is larger.
- On a bedding layer that consists of a clean granular soil, has a minimum thickness of 8 inches, and is only compacted outside the middle one-third of the width.
- In the granular backfill soil so that it experiences peak deflections on the order of 0.2% to 0.5% for the PVC pipes and 1% to 2% for the HDPE pipes.

## 2.3 Pennsylvania's I-279 Deep Burial Study

### 2.3.1 Background

This section summarizes the I-279 deep burial study performed by the Pennsylvania Department of Transportation (Goddard 2008). The project began in 1986 during construction of Interstate 279 from the north side of Pittsburgh to connect with Interstate 79 outside Wexford (Figure 2-4). This twelve mile highway had several cut and fill sections, varying from 40 feet to over 100 feet. With the help of Advanced Drainage Systems, Inc. (ADS) PennDOT initiated a research project that investigated the 20 year performance of HDPE pipes. These pipes were installed along I-279 during its construction. The research, led by Dr. Ernest Selig of the University of Massachusetts, resulted in new information on the structural integrity, deflection, and overall performance of these HDPE pipes.

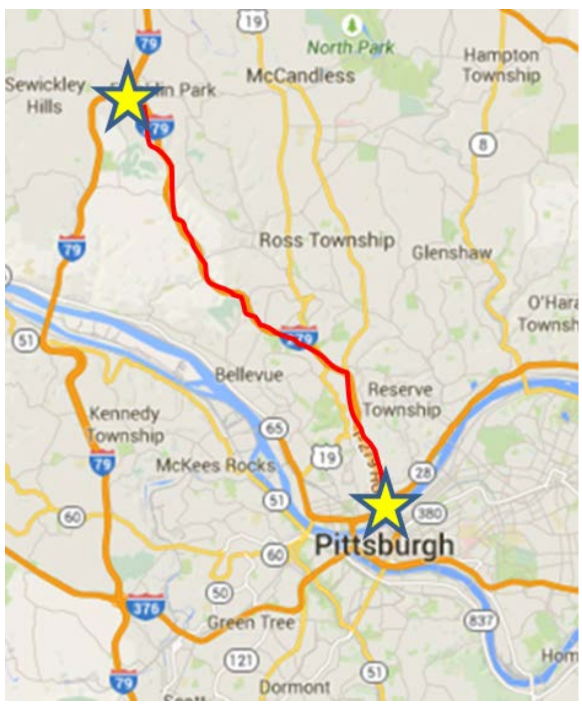



**Figure 2-4: Map of I-279, North of Pittsburgh Pennsylvania (Google Maps)**

2.3.2 Construction

Pipe installation began on August 4, 1987. 24-inch corrugated HDPE pipes were installed underneath and perpendicular to the highway. The total pipe length was 576 feet, with 200 feet of Type S smooth interior pipe, and the following 376 feet of Type C single wall corrugated inside and out. Wrap-around split collar couplings were used to connect the pipes. The maximum cover depth was over 100 feet. The pipe was manufactured to meet the AASHTO M294 specification of the time. (Goddard 2008)

**Figure 2-5: East End of Type C HDPE Pipe (Goddard 2008)**

2.3.3 Pipe Deformations and Performance

When the pipe was inspected in 1990, three years after its construction, cracks were discovered. These cracks were found in the Type C pipe under the couplings where the pipe was under 70 feet or greater of fill (Figure 2-6). Over the following four years, these cracks were monitored and researchers found that even though some cracks lengthened, they never extended outside of the couplings. Even though the Type C pipes experienced cracking under their couplings, the Type S pipes, even at 100 feet, did not crack. Goddard explains this failure when he stated that “the Type C coupling fit into the corrugation valley like a wedge, putting the corrugation into tension across the inside corrugation valley.” Researchers predicted that cracking under the couplings would begin to appear in the Type C pipe under less than 70 feet of cover as time increased; however, cracks were not discovered during the entire twenty year study. No cracks were found in pipes with less than 70 feet of cover; cracking was only found within the Type C piping with 70 feet of fill and greater underneath the couplings.

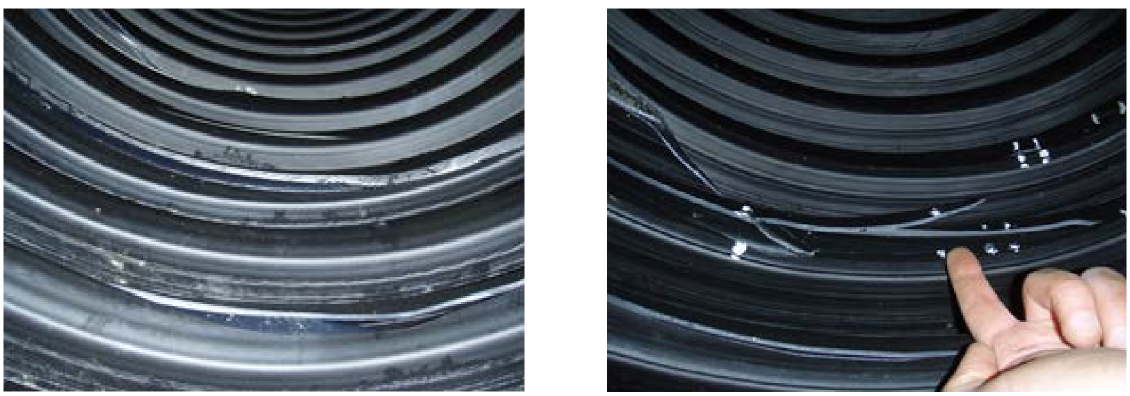


**Figure 2-6: Cracking Discovered under Couplings (Goddard 2008)**

Deflections in the pipe did not change significantly. The small variations in deflection and shape were attributed to different measurement techniques and locations. Researchers used a tape measure in the original measurements while later measurements were taken with a spring-loader LVDT.  Two oxidation induction time tests were conducted on the piping material installed at the site. The first was performed in 1997, when a sample was taken from the exposed pipe on the western side. The second was performed in 2007, when samples were taken from the exposed pipe ends on both the western and eastern sides. Samples shown in Figure 2-7 are circular with a radius of 1.5 inches. The results of these tests found no sign of degradation or oxidation on either side of the samples.

**Figure 2-7: Samples taken for Oxidation Induction Time Tests (Goddard 2008)**

2.3.4 Pennsylvania's I-279 Deep Burial Study Summary

The pipe installed in the I-279 research project held its structural integrity. There were no structural failures or excessive deflection. Total vertical deflection shortened by less than 5% over the 20 years in service. The deflection or out-of-roundness was less than 3.5%. The overall shape of the pipe was slightly elliptical, with some flattening of the invert of the pipe due to the firm bedding. The shape and strains has not significantly changed from 1992 to 2008, and no residual stresses were found in the tests performed.

The anti-oxidant additives were still functioning to standards when last measured in 2008. Levels were lower in the fully exposed pipe ends. Slight differences were shown between the invert and the crown. This difference is still within the test variability. The crack resistance of the Type S pipe resin after 20 years nearly met current requirements of 24 hours at 600 psi constant ligament tensile stress. Crack resistance testing samples for Type C pipe were taken from the exposed ends and even though these values are slightly lower, they are still over the current specification requirements. The only cracks found in the pipe exist underneath couplings of the Type C pipe with cover of 70 feet and up. These cracks did not grow outside the covering of the couplings and stopped lengthening in the last 10 years of the study.

# SECTION 3

# ALABAMA SITE INVESTIGATIONS

## 3.1 Background

On 13 July 2015 the ALDOT County Transportation Bureau forwarded an email developed by the research team to all Alabama County Engineers asking for assistance:

> *One goal of the current "Phase 2" project is to study the performance of pipes that were installed under normal conditions, rather than as part of a research project. Therefore, we need the assistance of county and municipal engineers in identifying candidate pipes. We are interested in any information and guidance that engineers can provide, but the general selection criteria are simply that the pipes are plastic (typically HDPE or PVC), installed under roadway, and have a 36-inch or greater diameter. In addition to visiting the site and documenting the condition, we will need to meet briefly with engineering representatives and ask a few basic questions about the installation.* (partial email)

Three counties (Clarke, Crenshaw and Jefferson) responded that they do not have any cross drains involving plastic pipes. The only counties that responded with potentially useful cases were Elmore and Colbert counties, which are discussed below. Additionally, the research team was already familiar with an HDPE installation in Opelika, so details of the condition of those pipes are provided, and ADS pointed the team to an installation in Montgomery, which is also documented below. The research team also discussed recent plastic experience with GDOT highway engineers, who pointed to seven test installations of ADS polypropylene pipelines installed in 2012 through 2014. Given that these GDOT installations were recent, there is no data or significant observations that can be reported herein, but a follow-up with GDOT within a few years may be useful.

## 3.2 Opelika

### 3.2.1 Background and Location

The site at the intersection of Rocky Brook Road and North Hills Drive in Opelika Alabama has two 60-inch HDPE pipes. This location is 10 miles northeast of Auburn University (Figures 3-1 and 3-2). Rockybrook Road is a local two-lane road that services several neighborhoods.  These two pipes travel underneath a two lane road connecting local residential areas and service a small creek. The current double-barrel HDPE configuration was installed as a replacement to the previous pipeline in the spring of 2005. The site originally contained a single 72-inch metal pipe that failed after major rain and flooding in the area. Details of the background and installation were provided by site visits and meetings with the City of Opelika managers and engineers.

The study of this pipeline was very important for several reasons. First, it was one of the only large diameter plastic pipes found that was installed as a cross drain. It was also important that the pipe had consistent daily traffic with an ADT of approximately 4500 vehicles. The last important factor was that the pipes were installed under construction practices and procedures that would be typical of contractors or department of transportation officials. These factors together give a good representation of the real world installation and performance of the specific factors requested in this study.

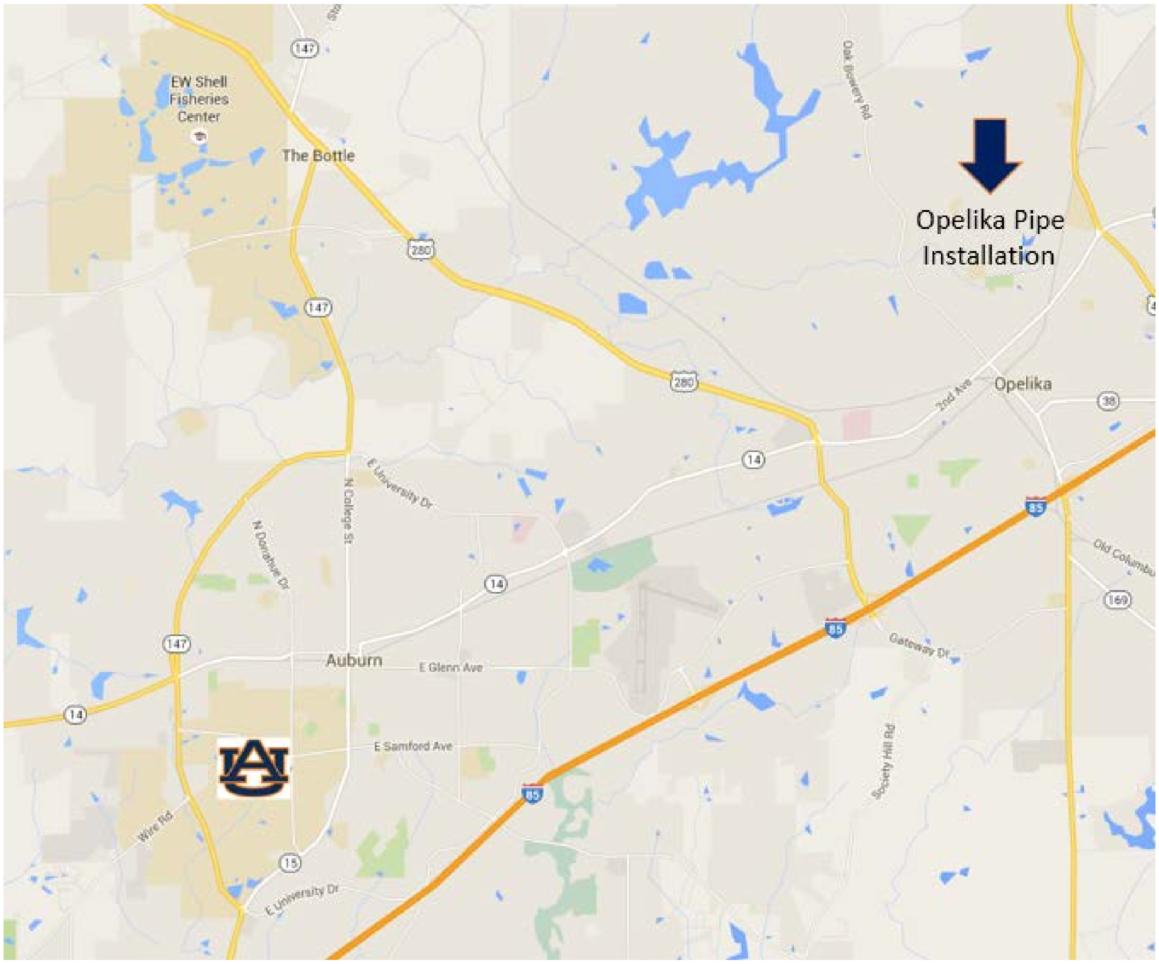



**Figure 3-1: Opelika Pipe Installation located Northeast of Auburn University (Google Maps)**


**Figure 3-2: Overhead View of the Opelika Pipe Location (Google Maps)**

3.2.2 Installation

The cross drain repair on Rocky Brook Road was an emergency repair and therefore had a rapid construction schedule. The installation was comprised of two 60-inch HDPE pipes that were approximately 240 feet in length and contained flared end sections. The pipes were backfilled with soil and stone aggregate. Two feet of #1 stone within a filter fabric were placed, then ten inches of 825-B crushed aggregate were laid before the final layer containing two inches of bituminous binder surface and the wearing surface. The construction project was done by local contractors.

Another heavy rain occured in the area midway through the repair project. This rainfall caused damage to the repair construction that had to be corrected before the project could be continued and completed. The selection of two large 60-inch diameter pipes was chosen so that the pipes could easily handle the water flow needed at this location.

#### 3.2.3 Pipe Inspections

##### *3.2.3.1 September 2008*

In September of 2008, the first Rocky Brook Road site inspection was conducted. Researchers entered the site checking on the overall condition of the pipe and surroundings. Pipe ends were examined for erosion or damage. Researchers then entered each of the pipelines to inspect the internal condition of the pipe. Deformations were found throughout locations of the pipeline. Pipe joints were also measured and checked for damage. No significant issues were noted with the pipe joints. Rippling deformation was observed within the sidewalls of the pipe at several locations.

##### *3.2.3.2 July 2015*

In July of 2015 researchers revisited the Opelika site to analyze its performance over the first ten years. Upon entering the pipes, it was observed that the large diameter of the pipes were sufficiently handling the volume of water flow. Each pipe end along with the concrete end caps were in good condition. Significant deflections were observed from looking along the length of each pipe. These deflections, depicted in Figures 3-3 and 3-4, show vertical diameter reduction and the expansion of the horizontal diameter causing an oval shape. This cross section contortion varied at different positions but continued along the entire lengths of each pipe. Measurements of the vertical and horizontal deflection were not taken at that time. Rippling damage, shown in Figures 3-5 and 3-6, was observed in multiple locations on the side walls of each pipeline. The rippling damage extent varied from approximately 3 feet to almost 7 feet in length and 2 feet in height. The joints were examined and damage or separation was not observed.

**Figure 3-3: Inside View of Pipe with Creek entering Pipe End**

**Figure 3-4: Large Internal Vertical Deflection**

**Figure 3-5: Rippling Discovered along Pipe Sidewalls**

**Figure 3-6: Rippling Damage in the Pipes Sidewall**

*3.2.3.3 Spring 2016*

During the spring of 2016, multiple visits were made to the Opelika pipe site to check the condition of the site and to see if any additional changes had occurred. The pipe ends and concrete end caps remained in good condition. When viewing the length of the pipe from one end, significant deformations could be easily seen (Figures 3-7 and 3-8). In order to start a more accurate procedure for tracking deformation changes, measurements were taken of the vertical and horizontal deflections. These measurements were also used for comparison of the original pipe dimensions. The results of these deflection measurements are shown below in Figures 3-9 and 3-10.

**Figure 3-7: Increased Deflections Occurring within Pipe A**

**Figure 3-8: Large Deflections Occurring in Pipe B**

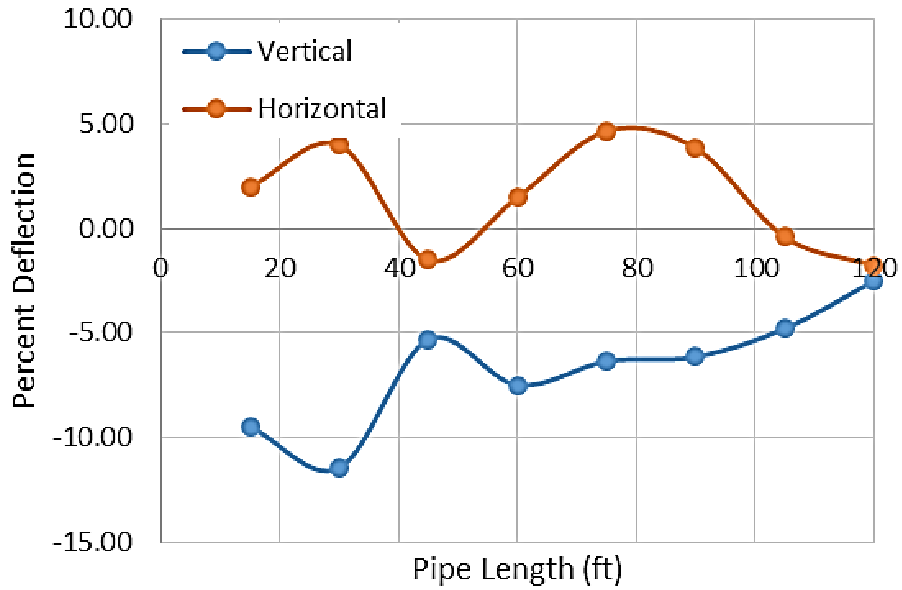



**Figure 3-9: Spring 2016 Percent Deformation in Pipe A**

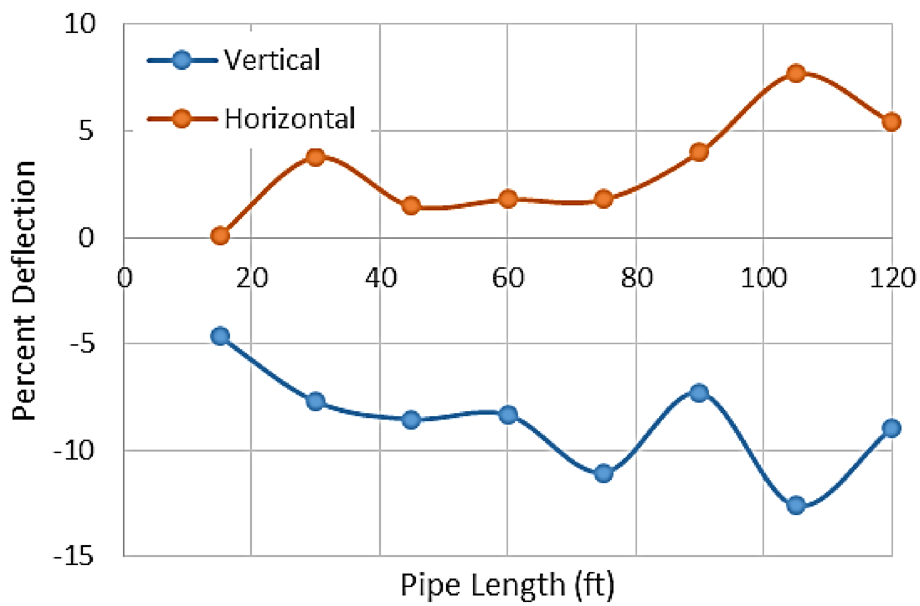



**Figure 3-10: Spring 2016 Percent Deformation in Pipe B**

While examining the internal segments of the pipes, it was clear that it had undergone increased damage over the previous year. In addition to the observation of increased deformation in the pipe diameter, significant joint damage was found. In the joint where two pipe segments were connected, a fragment of the pipe end had torn and was hanging (Figure 3-11). An issue in the joint was also found in the other pipeline. At this location, shown in Figure 3-12, separation had begun between the connection of the two pipe segments. The rippling damage (Figure 3-13) previously found appeared to have remained the same length and height over the year time period.

**Figure 3-11: Tear in the Pipe End at a Joint Location within Pipe A**

**Figure 3-12: Joint Separation**

**Figure 3-13: Continued Rippling throughout the Length of both Pipes**

### 3.2.4 Opelika Pipe Analysis

Large deflections were observed throughout the pipe, along with rippling in the sidewalls. These two factors were not hindering the overall performance of the pipe but contribute to long term concerns. When visited a year later, the site conditions had visibly worsened. In only a year between inspections, the pipe showed new damage and separation in the joints.

Deflection measurements demonstrated large the deformations over a relatively short time period (ten years). The vertical deformations in the two pipes ranged from -12% to 3% and -13% to -4%. The horizontal deformations in the two pipes ranged from -2% to 5% and 0% to 7%.  These measured deformations were significantly large and are likely to continue to grow throughout the pipes service life.

The limited knowledge about the installation procedure should be noted. It is not known to what extent the installation procedure and rain damage during the repair construction affected the final product. Since this pipeline was installed by typical workers, it can be assumed that the procedure would likely be performed by other construction groups installing similar lines.

## 3.3 Montgomery

### 3.3.1 Background and Location

An installation with three 36-inch HDPE pipes was investigated in a rural area outside Montgomery Alabama in November of 2015 (Figure 3-14). This location was brought to the attention of researchers by Advanced Drainage Systems, Inc. (ADS). The three pipelines were placed underneath a two lane road that connected residential and commercial buildings (Figure 3-15). The road paralleled train tracks. The pipes service storm water runoff in the area. Information on the construction and installation procedures was not available. The three pipes were approximately 30 feet long and buried at a depth of approximately two feet beneath the roadway surface. The age of the installation was not known but was estimated to have been done within the last five to ten years. The ADT for this site was estimated to be less than 500 vehicles per day.

**Figure 3-14: Three 36-inch HDPE Pipes in Montgomery**

**Figure 3-15: Two Lane Road Traveling over Montgomery Site**

### 3.3.2 Inspection

The pipes were dry upon inspection. At each pipe end were numerous shrubs and grasses that covered the pipe openings, which were removed for the inspection. Each pipe end and concrete end caps were in good condition. The pipes were straight and did not indicate any significant vertical or horizontal deformations (Figure 3-16). Measurements taken along each of the pipelines confirm this observation. The vertical and horizontal diameter measurements throughout the pipes were almost all exactly 36 inches throughout. Only a few measurements were not 36 inches and skewed from this original diameter of only a quarter inch. Pipe joints were inspected and no damage or separation was noted.

**Figure 3-16: Internal Conditions of Center Pipeline**

3.3.3 Montgomery Pipe Analysis

The HDPE pipes inspected in Montgomery showed no signs of stress. Deformations were not significant and the joints did not indicated any problems. The plastic piping materials used in this cross drain were performing correctly. The site location was given to researchers by ADS, and little is known about installation time and installation practices. The site should continue to be monitored to see if conditions change as the pipe ages.

## 3.4 Elmore County

### 3.4.1 Background and Installation

In the city of Wetumpka, which is located northeast of Montgomery in Elmore County, county engineers informed researchers of a plastic pipe installation placed underneath a service road behind one of their county engineering facility buildings. The site has a single 36-inch polypropylene pipeline (Figure 3-17) that was installed in 2010 by county engineers using pipe installation procedures used in the county. This included a bedding of #4 stone before it was filled with a native clay and gravel mixed soil in 6 to 8 inch lifts. These lifts continued until the three feet of cover was completed. Compaction for each lift was performed with a 54-inch Dynapac. The pipe was placed underneath a service road used by county construction vehicles traveling from the facility buildings to the surrounding public roads (Figures 3-18 and 3-19), which resulted in a lower ADT of only construction vehicles. Even though the ADT was low, these vehicles induced large loads onto the pipeline.

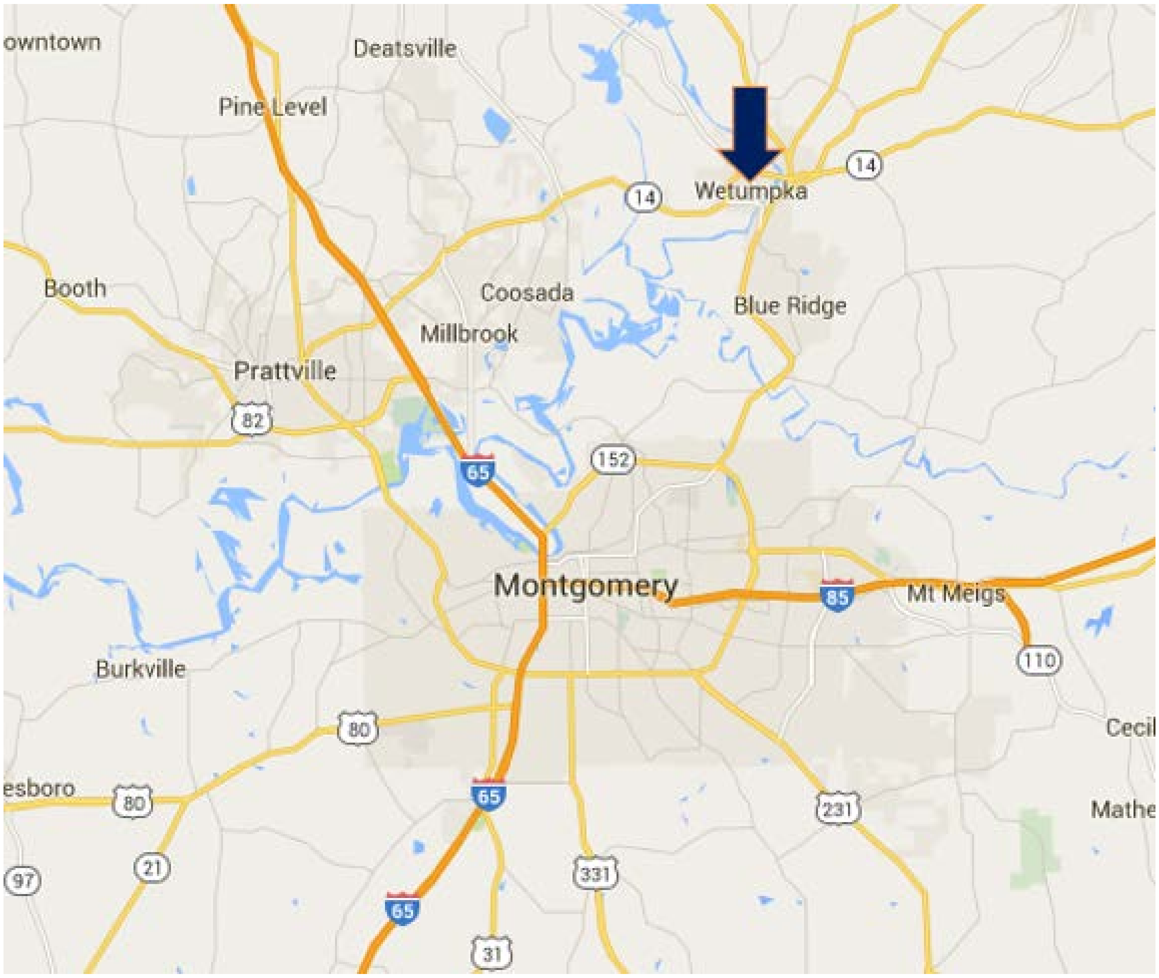



Figure 3-17: Location of the Wetumpka Installation Site (Google Maps)

Figure 3-18: 36-inch Diameter Pipe underneath County Service Road

**Figure 3-19: Service Road chosen for Pipe Installation**

3.4.2 Pipe Analysis and Conclusion

The analysis of the Wetumpka site began with inspection of the pipe ends. Each pipe end was in good condition. The pipeline did not have concrete end caps. The soil surrounding each pipe end showed no signs of erosion. While examining the inside of the pipe, no significant vertical deformations were noted. No separation or damage could be found in any joint; pipe joints appeared to be in good condition. Figure 3-20 provides an overall view of the pipe interior.

**Figure 3-20: Internal Inspection of Wetumpka Pipeline**

The Wetumpka installation site involves a plastic pipe that was a successful cross drain when inspected in its first six years of installation. The fact that the pipe was constructed under typical county installation practices was also a good factor to support the plastic pipe usage under proper conditions. However it is subjected to low ADT of only heavy county construction vehicles. It should be monitored into the future to be able to make conclusions about its long term performance.

### 3.5 Colbert County

Colbert County installed three lines of 48-inch double wall HDPE pipe in 2014. The installation replaced a double run of metal pipes that had failed from corrosion and was done by county forces. The pipes are depicted in Figures 3.21 and 3.22. As can be noted, the pipes are shallow buried and on a low ADT rural roadway.

**Figure 3-21: Colbert County HDPE Pipeline Side View**

**Figure 3-22: Colbert County HDPE Pipeline Front View**

### 3.6 Alabama Site Investigations Summary

Alabama county engineers were contacted with ALDOT assistance to inquire about plastic pipe installations in their jurisdictions.  Only two counties responded that they have installations that fit the criteria: Elmore and Colbert counties.  Both of those scenarios are evaluation cases being closely monitored by county forces.  Another double-line set of pipes were inspected in Montgomery County that serves a low ADT roadway. At the time of the research team inspections and contacts, there were no significant observable problems with the pipes.  In contrast however, a double line of large diameter HDPE pipes in Opelika were carefully inspected on several occasions that are indicating significant distress within less than ten years of service.

The use of thermoplastic pipes as cross-drains under highways in Alabama is still very limited, but the inspections discussed in this section support the conclusion by the Phase 1 project that, although it is likely that thermoplastic pipes may be suitable under certain conditions, their long-term performance is highly dependent upon the implementation of proper construction procedures and post-construction verification.

# SECTION 4

# BEEHIVE ROAD FIELD MONITORING

## 4.1 Background

A field test site was crucial to establish a first-hand deeper knowledge on the performance of plastic pipes. ALDOT responded to this need by sponsoring the construction of a site where pipes of different thermoplastic materials, geometries, and backfill conditions could be installed under real world conditions. The location chosen for the installation was underneath Lee County Road 12, (Cox Road, but referred to herein as the Beehive Road site) just south of Interstate 85 in Auburn, Alabama (Figures 4-1 and 4-2). Other test sites performed throughout the United States have been in northern regions where temperature, rainfall, and other natural conditions differ. The decision to have a local test site is intended to allow researchers to monitor the performance of pipes under conditions present in the state of Alabama.

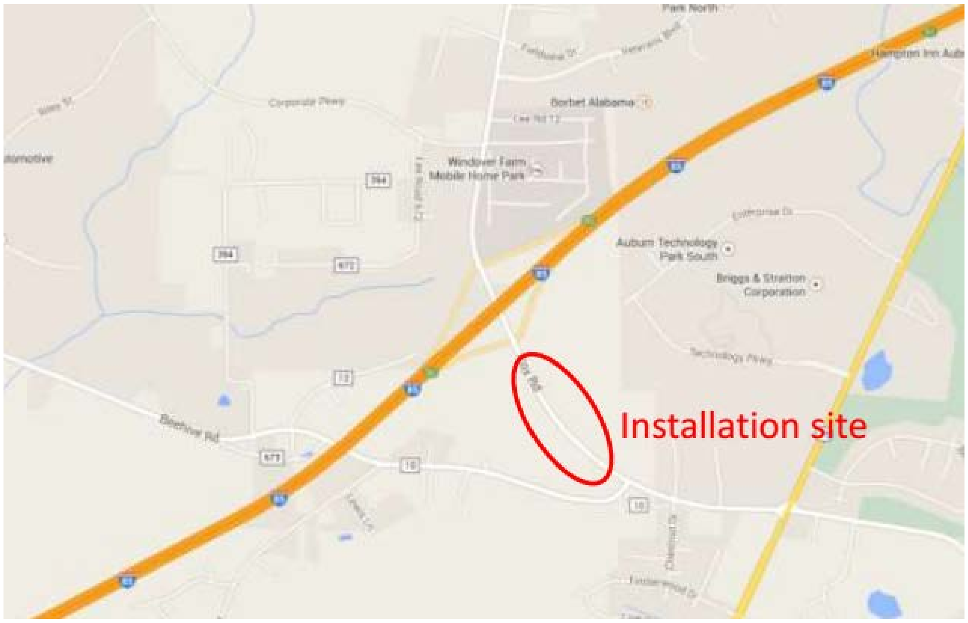



**Figure 4-1: Installation Site at Beehive Road South of Interstate 85 (Google Maps)**

**Figure 4-2: Thermoplastic Pipes Installation Location (Google Maps)**

## 4.2 Construction

### 4.2.1 General Construction Procedures

Construction on the Beehive Road research project began in December of 2010. Five thermoplastic pipe lines were installed, illustrated in Figure 4-3, running perpendicular underneath the four-lane highway. These pipe installations were placed to investigate the performance of these pipes in cross drain applications and were not required to drain any water. Each pipe run had differing pipe diameters, fill heights, and bedding and backfill materials. Observations could then be made how each of these variables affected the overall performance of each pipe. A more detailed report about exact construction details can be found in the ALDOT Project 930-718 report titled "Evaluation of HDPE and PVC Pipes Used for Cross-drains in Highway Construction" (Stuart et al. 2011).

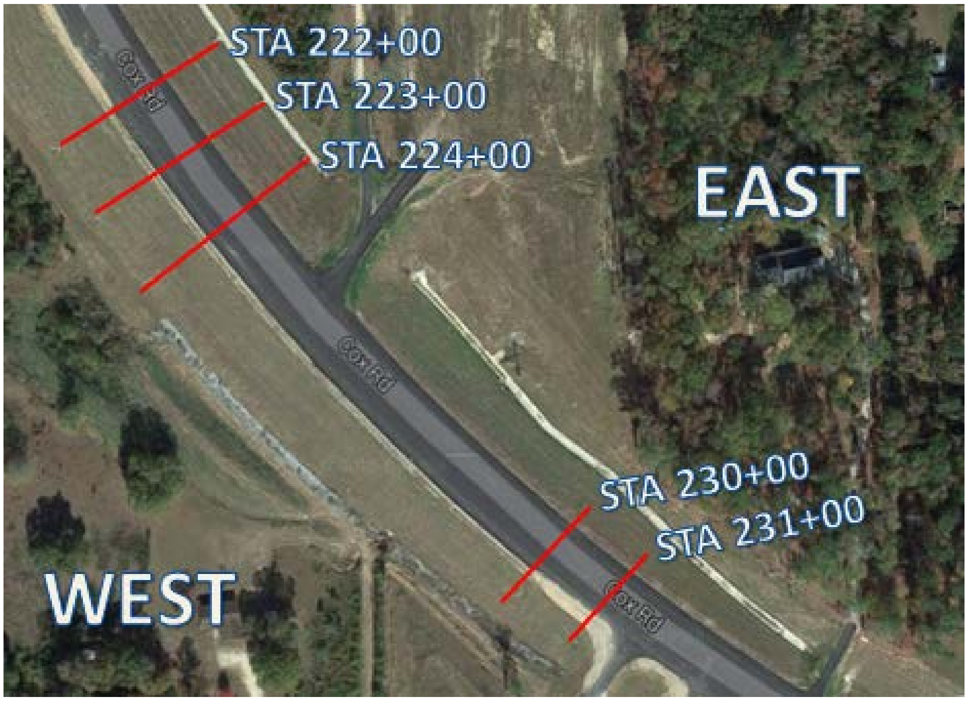



**Figure 4-3: Positions of the Five Thermoplastic Pipe Runs (Google Maps)**

The five pipe runs were constructed with the following installation details, with each pipe run labeled and distinguished by its own station number. The first three stations, Station 222+00 through 224+00, contain pipe runs that are split in the middle. On each half of the run is two differing pipes distinguished by the piping material, diameter, or backfill. For the pipelines that are split, the pipes were connected in the middle using a cast-in-place concrete junction box. The last two stations, Station 230+00 and Station 231+00, are roughly half the overall length as the first three. Along with having a much shallower maximum cover of only 8 feet, these pipe runs are not split in the middle and contain consistent pipe properties throughout the entire length. Every pipe end was installed with a standard concrete end treatment with a 4:1 slope. It was not intended that the pipes would serve any drainage purpose.

Station 222+00 is the northern most pipe run installed and is 254 feet long. It contains two different types of piping materials. The eastern half contains a 36-inch diameter PVC pipe under class III backfill and the western half contains a 36-inch diameter HDPE pipe under class I backfill. Station 223+00 follows this same split in the middle, with two different piping materials on each half under the same class II backfill. The eastern half also contains a 36-inch PVC pipe and the western half containing a 36-inch HDPE pipe. Stations 224+00 through 231+00 all contain the same piping material but of different diameters on each half. HDPE pipe was installed along the entire run of Station 224+00, with a diameter of 48 inches on the eastern half and 54 inches on the western half all under class I backfill. Station 230+00 contains a 36-inch diameter PVC pipe that runs the entire length of the run under class III backfill. Station 231+00 has a

36-inch diameter HDPE pipe that runs 128 feet under class III backfill. The details of each pipe run installed are summarized in Table 4-1.

**Table 4-1: Thermoplastic Pipe Installation Details (Stuart et al. 2011)**

| | | | PVC | | HDPE | |
|---|---|---|---|---|---|---|
| STA | COVER | LENGTH | DIA | Backfill | DIA | Backfill |
| 222+00 | 25-28 ft | 254 ft | 36" | Class III | 36" | Class I |
| 223+00 | 26-30 ft | 258 ft | 36" | Class II | 36" | Class II |
| 224+00 | 25-28 ft | 260 ft | N/A (entire length HDPE) | | 48"/54" | Class I |
| 230+00 | 4.5-8 ft | 128 ft | 36" | Class III | N/A (entire length PVC) | |
| 231+00 | 4.5-8 ft | 128 ft | N/A (entire length HDPE) | | 36" | Class III |

These pipe installation details were examined and agreed upon by researchers, ALDOT engineers, and representatives from the pipe manufacturing companies. All HDPE pipes were manufactured by ADS (Advanced Drainage Systems, Inc.) and all the PVC pipes were manufactured by Contech (Contech Engineered Solutions, LLC). The soil used in the construction was the lowest quality allowed by manufacturers' specifications for each burial depth. Researchers could then observe the pipes long term performance under the most lenient requirements. It was envisioned that these properties would yield an accurate portrayal of the long-term performance of plastic material pipes under differing installment details.

The construction and installation of pipes at the Beehive Road site was observed and documented by AUHRC researchers. The construction procedures were thoroughly documented and described in the ALDOT Project 930-718 Report (Stuart et al. 2011). This construction performed between December 2010 and January 2011 is summarized in the following sections.

### 4.2.2 Station 224+00 Construction

Construction on the site began with Station 224+00 on December 21, 2010 with the installation of the 54-inch HDPE pipe. The weather conditions were sunny with temperatures ranging from 50 to 63 degrees Fahrenheit. The construction process began with the digging of the trench starting from the downstream end using an excavator. The trench formed ranged from 8 to 10 feet and included a step. The step, illustrated in Figure 4-4, was added to avoid an excessively tall and steep trench wall that would pose a threat to safety. The trench was dug for one pipe section at a time. This meant that only about 25 feet of the trench was open at one time. For each open segment, a Grade Light 2500 was used to set the line and grade of the trench bottom.

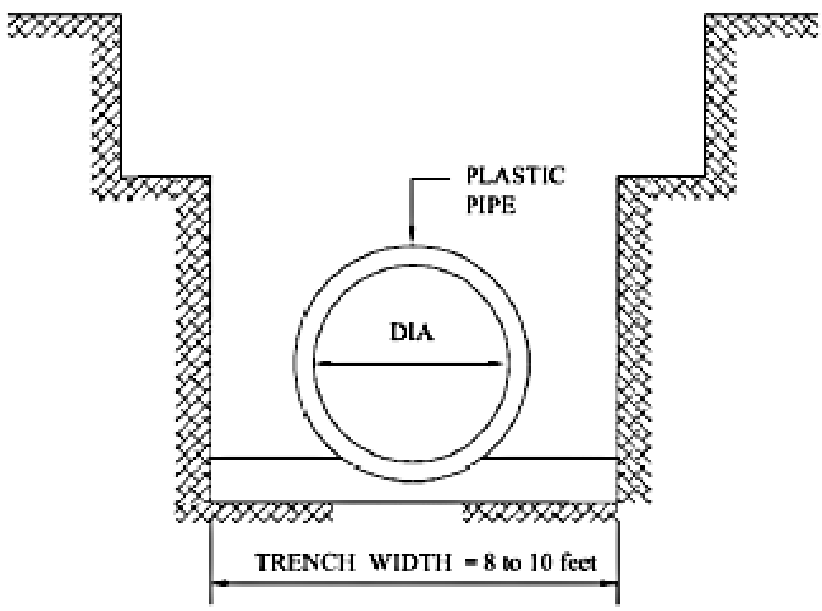



**Figure 4-4: Stepped Trench used for 54-inch Diameter Pipe in Beehive Road Installation (Stuart et al. 2011)**

Once the grade of the trench bottom was set to the desired 1.46%, a 6-inch layer of bedding using #57 stone was laid along the bottom of the trench. A cradle was then formed within the bedding so that the pipeline could be lowered and set into the trench with the bell-end facing upstream. Pipe segments were joined together using a bell and spigot system. This system, shown in Figure 4-5, involves each pipe segment having the spigot end lubricated before it is then pushed in by an excavator that held the pipe at third points using nylon slings.

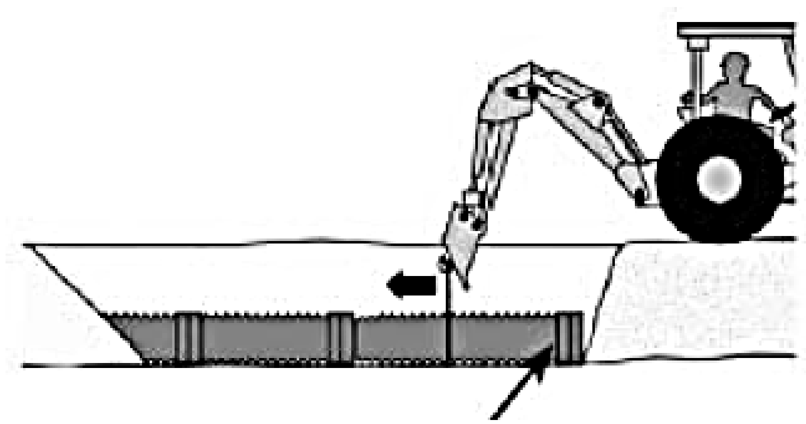


**Figure 4-5: Joining Pipes using the Sling Method (ADS 2010)**

The line and grade was then checked again before backfill was placed. The backfill was placed in lifts of approximately 8 to 16 inches before it was compacted using a Waker WP 1500 plate compactor. This backfill was carefully dumped over the middle of the pipe, so that it would spread evenly to both sides. The uniform backfill would ensure that the pipeline remained straight and would not roll or slide to one side of the trench. When backfill material was placed in the haunch region, special attention was given to ensure that the backfill was sufficiently compacted and would provide the needed support to the pipe in the haunch region. The compaction was completed by using a shovel to hand compact the stone under the bottom side of the pipe.

The lifts were continuously added until the stone reached approximately one foot above the top of the pipe. Above this point, standard native backfill ranging from 25 to 28 feet was used and compacted as standard embankment. A total of 125 feet of 54-inch HDPE pipe was installed on the western half of this station before it was attached to a concrete junction box in the middle of the pipeline.

On December 22, approximately 132 feet of 48-inch HDPE pipe was installed on the eastern half of this station using the same process and backfill material as the 54-inch HDPE installed at this station. The last section of the 48-inch HDPE pipe had to be cut in the field so that it would meet the required total length. Even with the shorter length pipe segment, no problems were observed during its installation into the pipeline.

An important note should be made about this station. The pipeline running across the project site intersected a major construction roadway running through the site. This resulted in the 54-inch HDPE pipeline being covered to provide a passageway for construction vehicles and equipment to cross the trench. Two feet of #57 stone and 4 feet of dirt backfill were placed above the pipe before any traffic was allowed to pass over. This cover was deemed sufficient by the manufacturer's representatives present on the site. Figure 4-6 displays a large dump truck crossing over the covered pipeline.


**Figure 4-6: Dump Truck Driving Over Pipeline (Stuart et al. 2011)**

4.2.3 Construction of Station 222+00

The installation at Station 222+00 began on January 14 and took two days to complete. The station was set to have a pipeline consisting of a western half of HDPE and an eastern half of PVC pipe. The final fill height for both halves was 25 to 28 feet with a grade of 0.46%. Construction began with the western half where 127 feet of 36-inch HDPE pipe were installed and then connected to the concrete junction box placed in the center of the station. The backfill material for the HDPE was ASTM Class I backfill which consisted of #57 stone. The HDPE pipe at this station had the same backfill as used in Station 224+00, and followed the identical installation procedure.

Construction then continued with the PVC pipe installed on the eastern half of this station. The backfill determined was ASTM Class III backfill, which was native yellow sand with gravel that was cut out of a different location of the construction site. This granular material followed a different installation process than the stone. Starting at the downstream end, a trench was dug for each pipe segment with a width of 7 to 8 feet. The bedding was formed with the Class III soil before the pipe was lowered and set into the trench. Backfill was then dumped on top of the newly installed pipe segment. This backfill, shown in Figure 4-7 would ensure that the pipe segment retained the correct grade and line while the next pipe segment was being attached.

**Figure 4-7: Backfill Placed to Secure PVC Pipe Segments (Stuart et al. 2011)**

Once the installed pipe segment was secured, the excavation for the next pipe section started. With the trench being dug and the bedding leveled, the lubricated spigot end of the next pipe was connected to the first pipe using the same excavator sling method as described before. The backfill was hand compacted with shovels into the haunch region. Backfill was then added in lifts of 12 to 18 inches. The plate compactor utilized in the previous station along with a jumping jack compactor was used to compact each lift. This process of adding lifts continued until the backfill reached one foot above the top of the pipe. Then standard backfill could be added to complete the desired fill height and complete the installation.

#### 4.2.4 Construction of Station 223+00

The construction at Station 223+00 began on January 15. This station consisted of 129 feet of 36-inch HDPE pipe that connected to the concrete junction box and 129 feet of 36-inch PVC pipe. This pipeline was set at a grade of .95% and was placed under 25 to 30 feet of backfill. ASTM Class II backfill was chosen for the entire line, which consisted of a sandy clay mixture. Construction workers began digging the trench with a width of 7 to 8 feet on the downstream side. A bedding of the Class II backfill, which was not compacted, was formed in the bottom of the trench for each pipe segment. Lifts of 12 inches were then added. Each lift was compacted by making two passes of both a jumping jack compactor and a plate compactor. The lifts were continued until they reached a foot above the top of the pipe. This installation process was used for both piping materials at this station.

### 4.2.5 Construction of Station 230+00

On January 17, construction began on Station 230+00. This pipeline was only half the length as the previous stations, and consisted of only one type of piping material and backfill type. A 36-inch PVC pipe that ran a total distance of 128 feet was installed. A grade of 0.5% and a fill height of 4.5 to 8 feet was established. The backfill chosen for this station was a Class III backfill, which differed from the Class III backfill used for the PVC pipe at Station 222+00. The backfill used at this station was a native soil that was used for the embankment of the roadway.

A trench with a width of approximately 8 feet was formed with a bedding of Class III backfill. This bedding was set to the correct grade before the pipe was lowered and set. When the first pipe was placed, #57 stone was poured on top of the pipe to secure it in place while the next pipe segment was attached. This was the same process that was described for the PVC pipe at Station 222+00. The backfill was then added in 12 inch lifts and compacted to manufacturer's specifications. Once the backfill reached the springline of the pipe the lifts ceased. A bulldozer was used to push and compact the rest of the backfill over the pipe until the total fill height was accomplished. The day after its installation, the pipeline was subjected to construction traffic traveling over the station as soon as the next day of January 18. This included large dump trucks filled with soil driving over five feet of backfill. Large clumps of dirt were formed within the backfill that could not be present when it was placed around the pipeline. This problem was solved when the large clumps were broken down into smaller pieces or discarded from the pile.

### 4.2.6 Construction of Station 231+00

On January 17, construction began on Station 231+00. This station consisted of a single line of 36-inch HDPE pipe that ran 128 feet. The backfill chosen was ASTM Class III native soil, which is the same as used in Station 230+00. Installation of this station was very similar to that of 230+00. A trench with a width of 8 feet was dug and bedding was formed in each trench with Class III soil. The first pipe segment was then lowered and set into the trench before it was secured by mound of #57 stone. The difference in this installation compared to the previous is that 12 inch lifts were added to the pipe until they reached one foot above the pipe. With the backfill was a foot above the pipe, a bulldozer was used to push the remaining backfill above the pipe and compact it until the desired fill height of 4.5 to 8 feet was achieved.

## 4.3 Pipe Performance and Deformations

Approximately 30 days after the installations were completed in January of 2011, the first post-construction inspections were initiated by researchers. The inspections were performed to evaluate the initial performance and deformation changes of the pipes after all construction was completed. Initial inspection included a variety of tasks. A mandrel was pulled through each pipe to verify diameter tolerances were met. Researchers also inspected the interior of each pipe to observe pipe deflection, distortion in the shape of the pipe, buckling or cracking in the wall surface, inlet end and outlet conditions, and finally any pipe joint openings or offsets.

Vertical and horizontal deflections of the buried pipes were tracked by direct measurement upon researcher's entry of each pipe. Direct measurements were taken at 10 foot intervals throughout the entire length of each pipe. These locations were marked with a paint pen so that future measurements would be taken at the same location. For the vertical measuring, the top mark was made first, then a plumb bob was used to accurately make a mark directly below the top mark. Horizontal marks were placed using a string and level. This ensured that the points were placed correctly and that measurements would be horizontal. With the marks made, a laser range finder was used to measure the distance between the marks. It should be noted however that over the years since the pipes were installed, different persons conducted the measurements, and therefore some variability will naturally occur due to inconsistencies in the manual recording methodology.

Each joint within the pipe was searched for openings or offsets. Measurements were taken at each joint so that changes could be observed over time. Sediment build-up and infiltration of joints was noted. The ends of each pipe were examined for erosion of the backfill around pipe. Pavement placed above the pipe installation was checked for damage or other integrity issues.

The measurements taken for the vertical and horizontal deflections are shown in graphs for each station based on the percent deformation. The percent deformation is calculated by taking the measured deformation compared to the manufactured nominal pipe diameter. For example, 48 inches was used for the 48-inch HDPE pipe. The Y-Axis consistently ranges from -5% to 5%. These minimum and maximum values were chosen based upon the fact that the target goal for thermoplastic pipes is to keep deformation within 5% of its original diameter. This 5% range comes from AASHTO M252 (2008) and AASHTO M294 (2008) for PE material pipe and AASHTO M304 (2007) for PVC material pipe. A positive percentage denotes that the pipes diameter has increased, while a negative percentage means the diameter has shortened. It should be noted that this positive and negative assignment to diameters may be different for data presented in previous reports.

### 4.3.1 Initial 30 Day Inspection

The findings of the initial 30 day inspection performed by researchers in January of 2011 are presented in detail in the ALDOT Project 930-718 report (Stuart et al. 2011) and are summarized below so that they can be compared to subsequent inspection data. Investigation tasks performed are listed and described within the opening paragraphs of Section 4.3 Pipe Performance and Deformations. Results and conclusions of these investigations are stated in the subsequent sections.

The initial post-construction inspection for the pipes at Station 230+00 and Station 231+00 was not completed until June of 2011. This delay was due to construction traffic crossing over the pipes and unfavorable weather conditions. A layer of mud that covered the bottom of the pipes was also present at these stations. This layer of mud had to be cleaned out before an accurate inspection could be made. Once these reasons for delay were resolved, the post-construction inspection continued.

*4.3.1.1 Initial 30 Day Inspection of Station 222+00*

The post-construction inspection of Station 222+00 was completed on January 30 of 2011. This station consisted of a 36-inch diameter HDPE pipe on the western half and a 36-inch diameter PVC pipe on the eastern half. During this initial inspection no major problems were found. Joint openings for the HDPE pipe ranged from no gap to 5/8 of an inch. Joints within the PVC pipe had no issues. Joints appeared to be sealed properly and no water or soil was infiltrating and settling within the pipe. The concrete end treatments at both the east and west ends showed no structural integrity issues or erosion.

Figure 4-8 summarizes the deflection data for the 36-inch HDPE pipe and Figure 4-9 displays the deflection data for the 36-inch PVC pipe. Vertical deflection in the HDPE pipe ranged from -2.6% to 0.4%. The horizontal deflection ranged from -3.8% to -1.2%. This largest horizontal deflection of 4% occurs at the very end of the length of pipe. For the PVC pipe, vertical deflections ranged from -2.3% to 1.4% and horizontal deflections ranged from -4.2% to 0%.

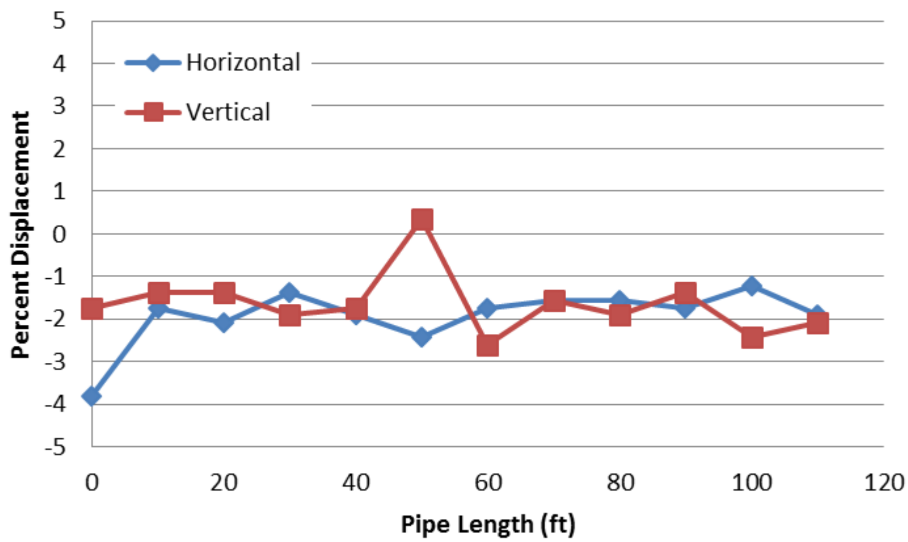



**Figure 4-8: Initial 30 Day Deflection Data for the 36-inch HDPE Pipe at Station 222+00**

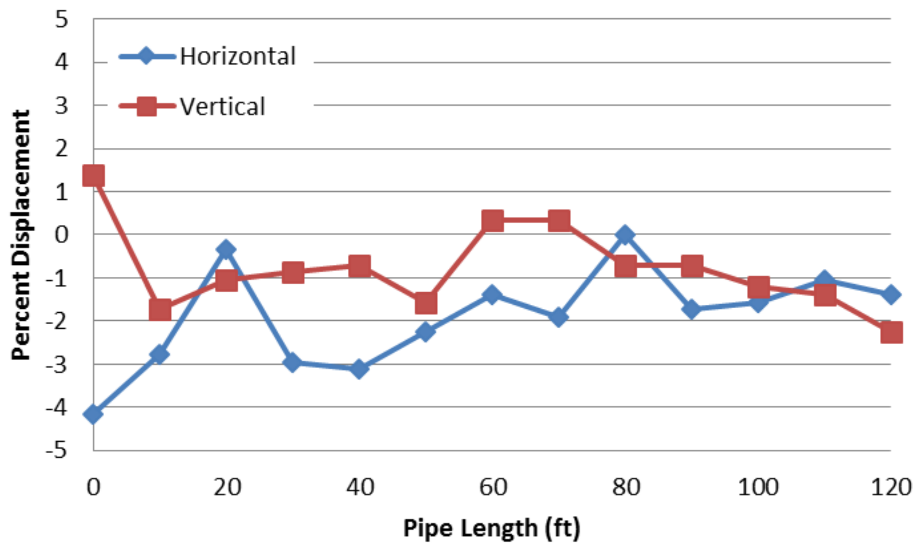



**Figure 4-9: Initial 30 Day Deflection Data for the 36-inch PVC Pipe at Station 222+00**

*4.3.1.2 Initial 30 Day Inspection of Station 223+00*

Post-construction inspection continued on January 30 with the pipeline installed at Station 223+00. This station contained a 36-inch diameter PVC pipe on the eastern half, followed by a 36-inch diameter HDPE pipe on the western half. No major problems were observed in the pipeline. Joint openings in the HDPE pipe joints were within tolerances, ranging from 1/4 of an inch up to 5/8 of an inch. PVC pipe joints were also performing up to standards and no problems were found. No water or soil infiltration of the pipe joints was found. End caps were free of erosion or other problems. Deflection data for the HDPE pipe is presented in Figure 4-10. For the HDPE pipe, vertical displacement ranged from -2.3% to -1.0% and horizontal displacements ranged from -5.0% to -1.0%. For the PVC pipe, vertical displacement ranged from -1.9% to 1.2%. Horizontal deflection ranged from -4.2% to 1.0% and is illustrated in Figure 4-11.

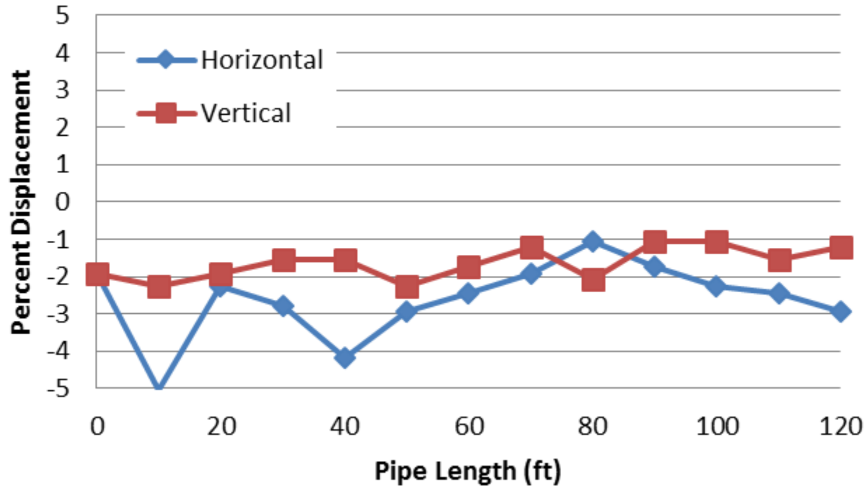



**Figure 4-10: Initial 30 Day Deflection Data for the 36-inch HDPE Pipe at Station 223+00**

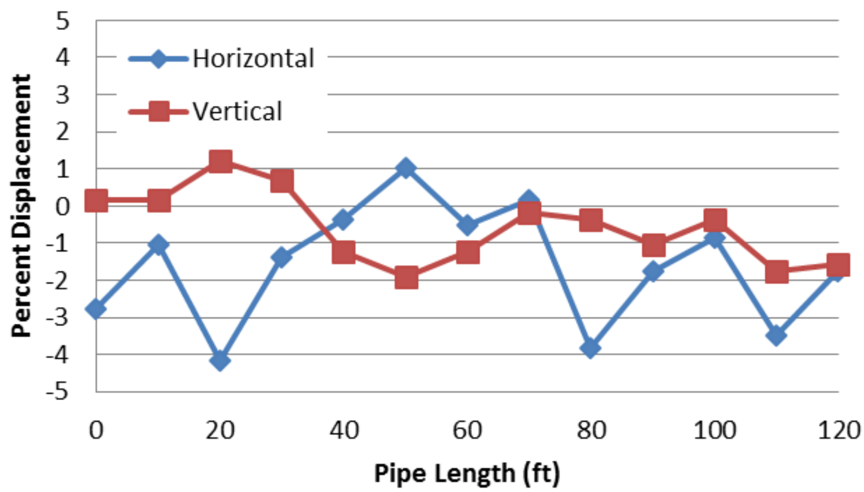



**Figure 4-11: Initial 30 Day Deflection Data for the 36-inch PVC Pipe at Station 223+00**

*4.3.1.3 Initial 30 Day Inspection of Station 224+00*

The initial 30 day inspection began with the review of Station 224+00. This pipeline consisted of a 54-inch diameter HDPE pipe on the eastern half, and a 48-inch HDPE pipe on the western half. No significant problems were found. Joint openings in the pipes at this station ranged from 0.4 to 2.5 inches. These openings, even at 2.5 inches were still within allowable limits due to the large bells on the HDPE pipes.

The deflection measurements taken within the pipe are summarized within Figure 4-12 and Figure 4-13. The vertical deflection of the 54-inch HDPE pipe ranged from -1.9% to 1.2%. The horizontal deflection for this pipe ranged from -1.3% to 1.3%. Deflection percentages for the 48-inch HDPE pipe were significantly larger. Vertical deflection ranged from -4.2% to -0.4% and horizontal deflection ranged from -3.9% to -0.5%.

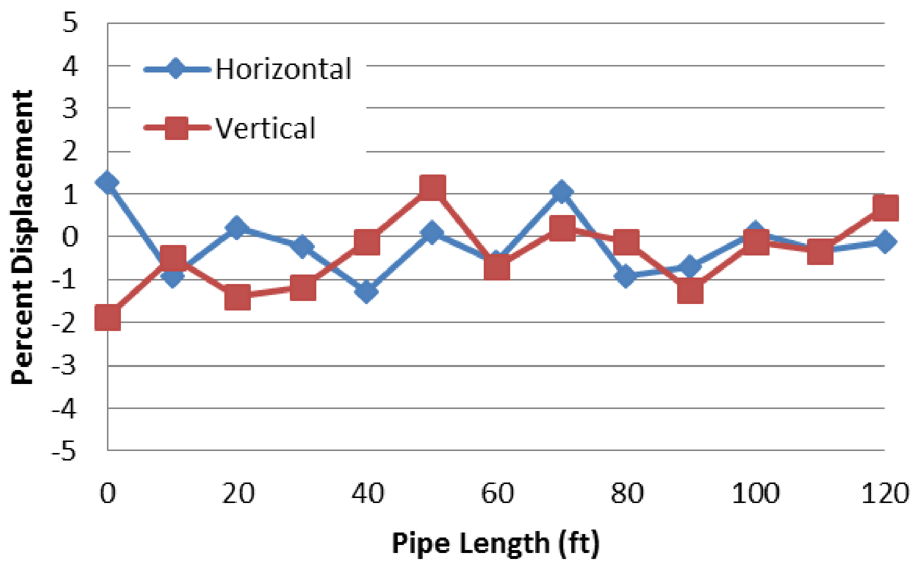



**Figure 4-12: Initial 30 Day Deflection Data for the 54-inch HDPE Pipe at Station 224+00**

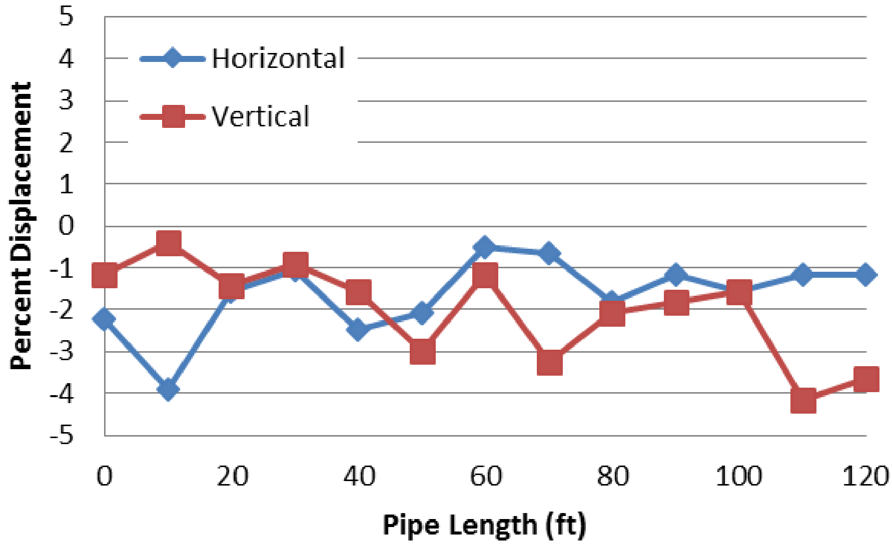



**Figure 4-13: Initial 30 Day Deflection Data for the 48-inch HDPE Pipe at Station 224+00**

*4.3.1.4 Initial 30 Day Inspection of Station 230+00*

Inspection of the 36-inch diameter PVC pipeline installed at Station 230+00 was completed on June 3 of 2011. No significant problems were observed during inspection. Joint openings in the PVC were tight and operating correctly. Issues with infiltration, settlement, or erosion were not apparent. Figure 4-14 illustrates that the vertical deflection ranged from -1.8% to 1.5%. Horizontal displacement ranged from -3.0% to -0.5%.

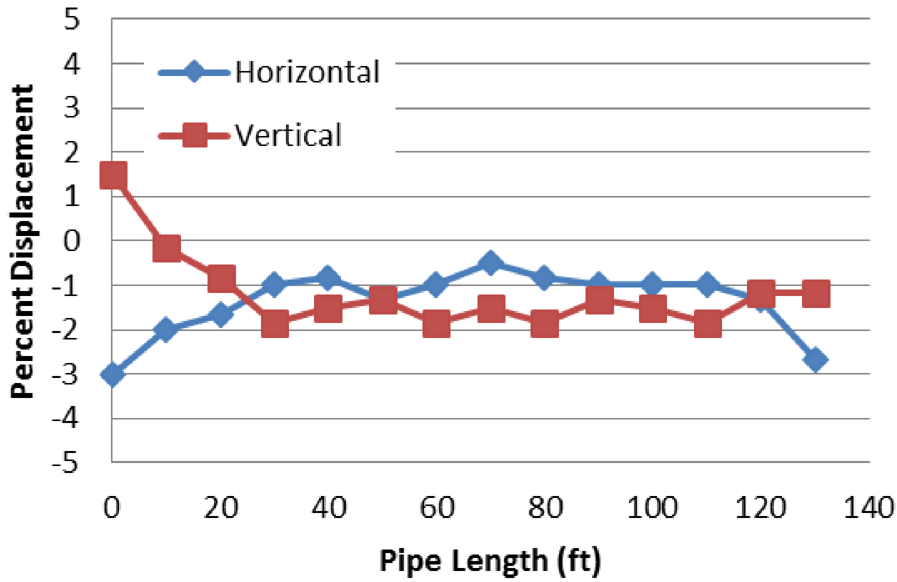



**Figure 4-14: Initial 30 Day Deflection Data for the 36-inch PVC Pipe at Station 230+00**

*4.3.1.5 Initial 30 Day Inspection of Station 231+00*

Inspection at Station 231+00 was also completed on June 3, 2011. No significant problems were found on this single line of 36-inch diameter HDPE pipe. Joint openings ranged from 1/4 inch up to 3/8 of an inch, which are within tolerance. No problems were found in the pipeline with infiltration, settlement, or erosion. As can be seen in Figure 4-15, vertical deflection ranged from -2.3% to 1.8%. Horizontal deflection ranged from -3.0% to 0%.

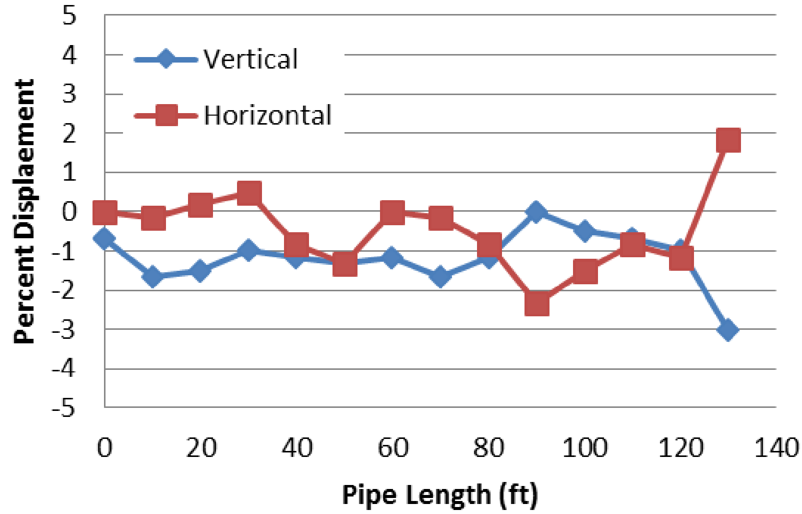



**Figure 4-15: Initial 30 Day Deflection Data for the 36-inch HDPE Pipe at Station 231+00**

### 4.3.2 Two Year Spot Check

Another inspection was performed in January of 2013. This two year inspection was limited and focused on deflection changes. The tasks performed during this inspection included taking vertical and horizontal deflection measurements at two locations within each of the pipes. These measurements were used to compare to the initial deflection data along the future inspections to define trends over the years after installation. Deflection measurements taken during this two year spot check are provided in Section 4.4 Analysis of Stations.

### 4.3.3 Four Year Inspection

On March 17 of 2015, another inspection was performed on each of the pipe installations. This inspection made by researchers was completed approximately four years after construction. The majority of tasks completed in the initial 30 day inspection were repeated. Researchers entered each pipe, inspecting the interior of each pipe to observe any distortion, buckling, or cracking. Vertical and horizontal deflection measurements were once again recorded at the same 10 feet increments as previously measured. Pipe joints were inspected for changes in joint openings and to examine if any soil or water infiltration was present. Pipe ends were checked for damage and erosion of backfill. The four year inspection results are displayed in this section according to each station number.

Before the research team entered the pipes, some preparation was required in order for effective data collection to occur. Entry protection, shown in Figure 4-16, was installed at the end of each pipe when the roadway was constructed. This entry protection had to be removed in some locations in order for research personnel to enter the pipes. Additionally, significant amounts of sediment had built up and were found along the end segments of the pipe (Figure 4-17 and 4-18). Sediment and vegetation that had grown along the pipes entrances was removed.

**Figure 4-16: Entry Protection Installed at Pipe Ends**

**Figure 4-17: Vegetation Growth at Culverts End**

**Figure 4-18: Water and Sediment Buildup within Pipe**

*4.3.3.1 Four Year Inspection of Station 222+00*

The four year inspection began with Station 222+00. Inspection of the pipe ends revealed no soil erosion around the end caps. The concrete end caps were also free of damage or chipping. Researchers noted that pipe joints were still performing without issues. There was no discernable cracking, rippling or buckling of the pipe walls. Vertical pipe deformation in the 36-inch PVC pipe ranged from -5.0% to 0.3%, while the horizontal pipe deformation ranged from -3.5% to 0.5%. Deformations in the 36-inch HDPE pipe ranged from -4.7% to -0.7% vertically, and -3.8% to 0.7% horizontally. These deflections are displayed in percent deformation in Figures 4-19 and 4-20.

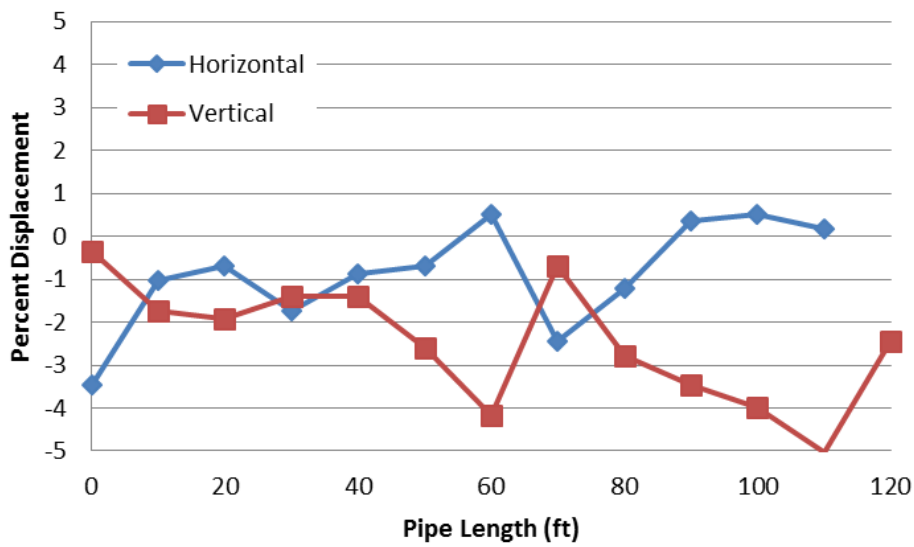



**Figure 4-19: Four Year Deflection Data for the 36-inch PVC Pipe at Station 222+00**

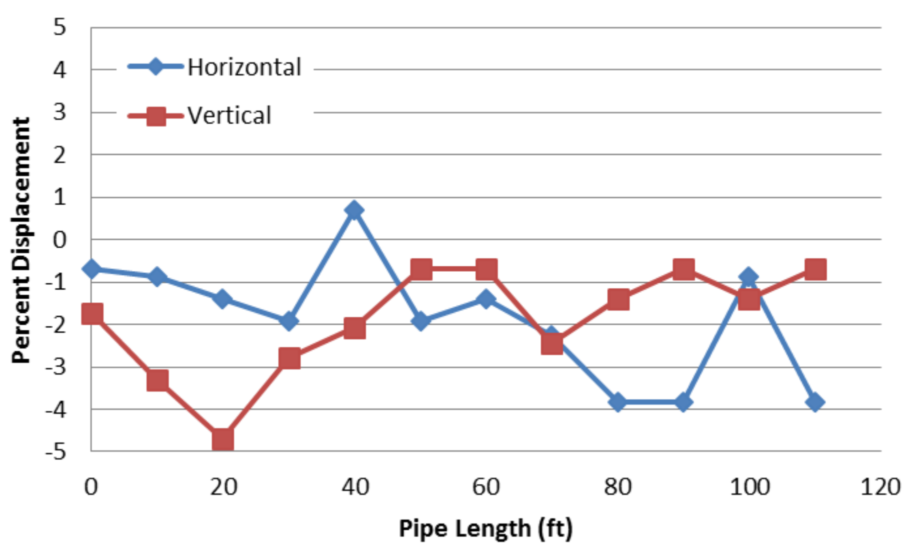



**Figure 4-20: Four Year Deflection Data for the 36-inch HDPE Pipe at Station 222+00**

*4.3.3.2 Four Year Inspection of Station 223+00*

Inspection of Station 223+00 began on the western half with the inspection of the 36-inch HDPE pipe. No major problems were found with the pipe end; the concrete end cap was in good condition and no erosion was observed. These findings were also true for the eastern end of Station 223+00. When entering the pipe from this western end, researchers came upon sediment up to 2 inches thick along with water that was pooling along the first 40 feet of the pipe. This prevented deflection measurements along this segment. Deflection measurements began after this 40 foot segment, where the pipe was once again clear. The vertical deflections of the HDPE pipe ranged from -3.7% to -1.6% and horizontal deflections ranged from -3.8% to 1.2%. These deflection measurements can be seen within Figure 4-22. No problems were found on the eastern end of this station, which contained the 36-inch PVC pipe. Joints and side walls were free of damage and separation. Deflections along this PVC segment were -2.4% to -0.7% for vertical deflection, and -4.0% to -0.5% for horizontal deflection. Pipe joints at this station were tight and performing satisfactorily.

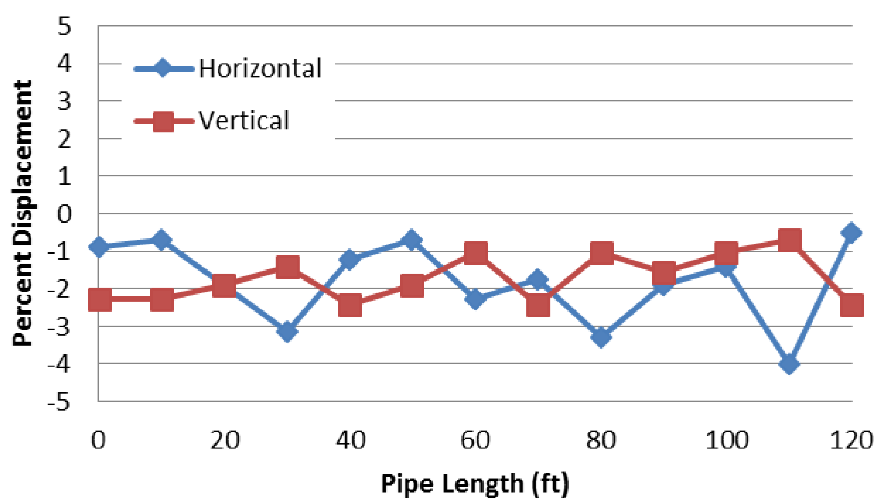



**Figure 4-21: Four Year Deflection Data for the 36-inch PVC Pipe at Station 223+00**

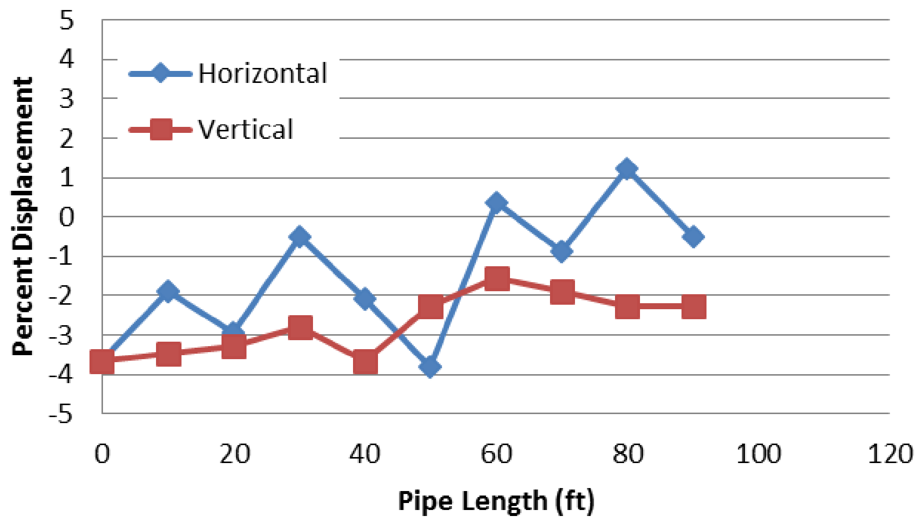



**Figure 4-22: Four Year Deflection Data for the 36-inch HDPE Pipe at Station 223+00**

*4.3.3.3 Four Year Inspection of Station 224+00*

Inspection continued with Station 224+00 that contained a 48-inch HDPE pipe and a 54-inch HDPE pipe. No problems were found in either pipe end. Joint openings remained relatively identical to the joint measurements made during initial inspection, which were within allowed ranges. It appeared that these joints were still sealed and no water or soil was infiltrating the pipe through these joints. Pipe deflections for both size HDPE pipes can be viewed in Figure 4-23 and Figure 4-24. Vertical deflections of the 48-inch HDPE pipe ranged from -5.2% to -0.8% and -5.7% to -1.0% for horizontal deflection. The two vertical deflection values at the end of this pipeline were -5.2% and -5.1%. The 54-inch HDPE pipe had deflections ranging from -3.7% to 0.9% vertically, and -1.9% to 0.4% horizontally.

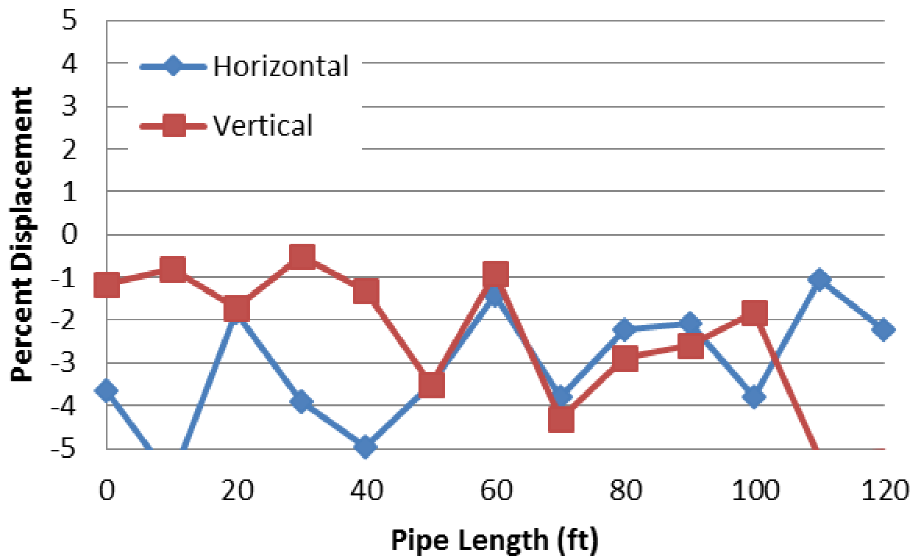



**Figure 4-23: Four Year Deflection Data for the 48-inch HDPE Pipe at Station 224+00**

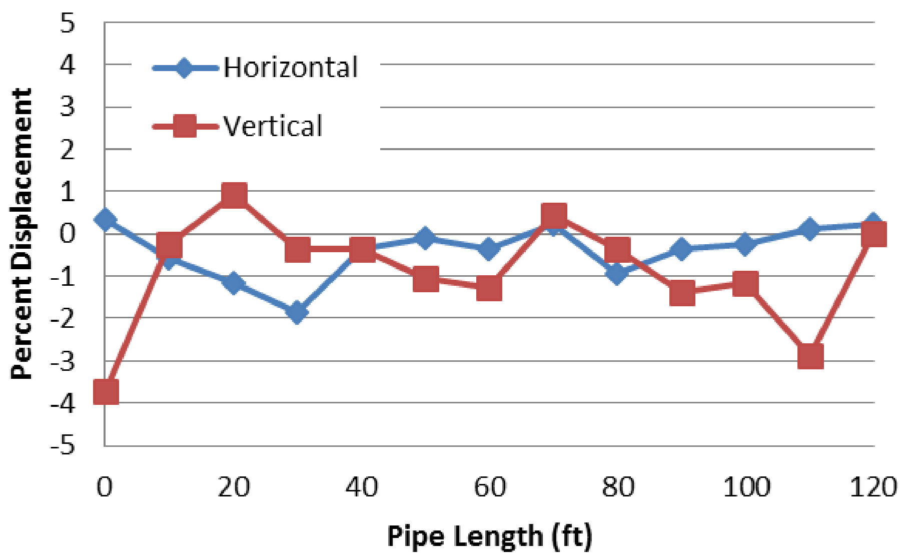



**Figure 4-24: Four Year Deflection Data for the 54-inch HDPE Pipe at Station 224+00**

*4.3.3.4 Four Year Inspection of Station 230+00*

No problems were found in or around the pipe ends of the 36-inch PVC pipe at STA 230+00. The joint openings were still within tolerance. The joints were in good condition and appeared to still be preventing any infiltration of soil. Pipe walls were free of major deformation, rippling, or buckling. The pipe deflection measurements taken for this station are illustrated in Figure 4-25. Sediment and water were found in the last 30 feet of this pipeline, which prevented researchers from performing measurements in this section. The vertical deformations measured ranged from -3.3% to 0.2%. The horizontal deformations ranged from -1.7% to -0.7%.

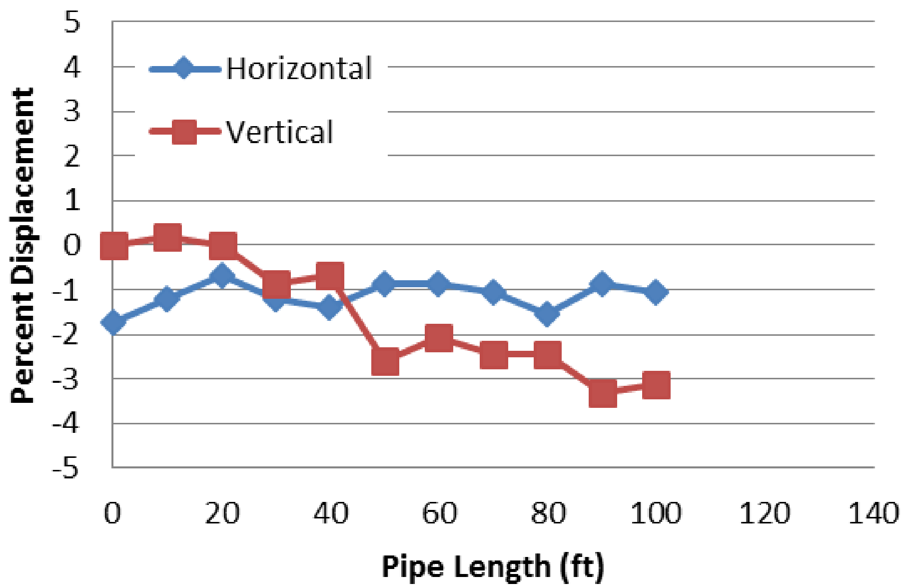



**Figure 4-25: Four Year Deflection Data for the 36-inch Pipe at Station 230+00**

*4.3.3.5 Four Year Inspection of Station 231+00*

The four year inspection was completed with the 36-inch HDPE pipe at Station 231+00. Pipe ends were in good condition. No erosion around end caps was found. Joints between each pipe segment were still performing properly. Pipe deflection data is shown in Figure 4-26. The vertical deformation measured ranged from -2.8% to 0%. Horizontal deformations ranged from -2.8% to 0%.

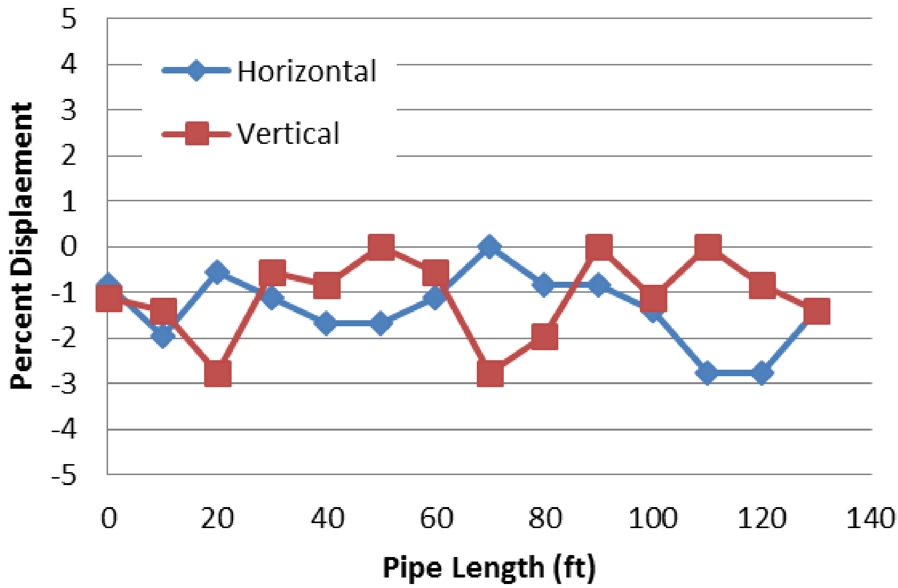



**Figure 4-26: Four Year Deflection Data for the 36-inch HDPE Pipe at Station 231+00**

4.3.4 Five Year Inspection

On January 13th 2016, inspection was performed on each of the pipe installations. These measurements were taken approximately five years after construction. The inspection tasks were identical to the tasks mentioned in Section 4.3.3 Four Year Inspection. Researchers entered each pipe, inspecting the interior of each pipe to observe any distortion, buckling, or cracking. Vertical and horizontal deflection measurements were once again recorded at the same 10 foot increments as previously measured. Pipe joints were inspected for changes in joint openings and to examine if any soil or water infiltration was present. Pipe ends were checked for damage and erosion of backfill. The five year inspection results are displayed below according to each station number.

#### 4.3.4.1 Five Year Inspection of Station 222+00

The five year inspection began with Station 222+00. The pipe ends were in good condition. No visible signs of damage or major problems to the pipe ends were observed. Minor erosion was evident in the soil surrounding the outside of the pipeline ends. When entering the pipe and viewing the entire length of the line, minor changes in the pipes deformations could be seen. Slight deformations were noted. Pipe joints were inspected and no signs of separation or soil infiltration were found. Large amounts of sediment were found in the bottom of the junction box in this station and in some portions of the HDPE pipe. The image in Figure 4-28 was taken two months after the five year inspection and shows the accumulation of sediment in the junction box. Vertical pipe deformation for the 36-inch PVC pipe ranged from 0% to -6.4%. The majority of these vertical deformation measurements were less than 5% but had two more extreme points near the junction box that were 5.5% and 6.4%.  Horizontal deformations for the 36-inch PVC pipe ranged from 2% to -4%. The percent deflection measured in the pipe segments are illustrated in Figure 4-27. Measurements for the 36-inch HDPE pipe at this station were not taken due to the amount of sediment and water within the pipe.

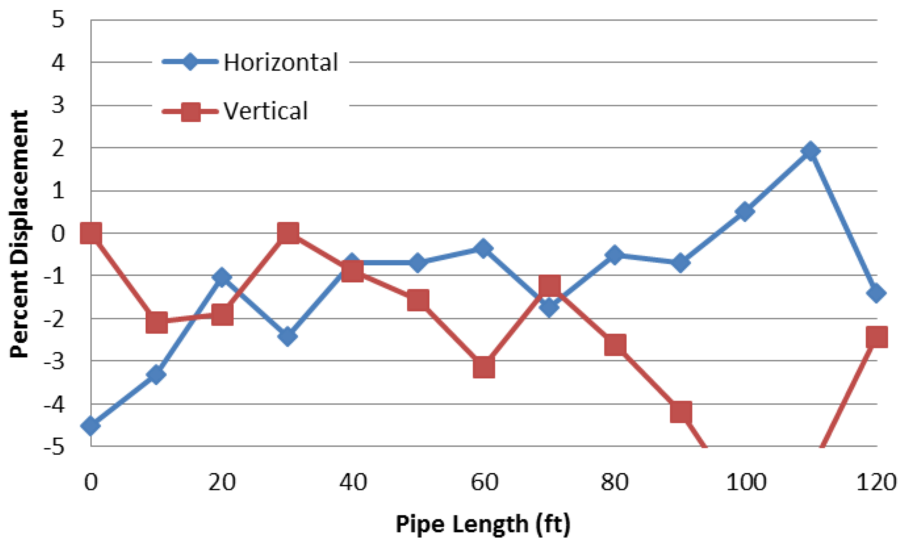



**Figure 4-27: Five Year Deflection Data for the 36-inch PVC Pipe at Station 222+00**

**Figure 4-28: Sediment Collected in Junction Box of Station 222+00 Pipeline**

#### *4.3.4.2 Five Year Inspection of Station 223+00*

The five year inspection continued with Station 223+00. Each pipe end was in good condition. No visible signs of damage or major problems to the pipe ends were found. Minor erosion was found in the soil surrounding the outside of the pipeline ends. Pipe joints showed no signs of separation or soil infiltration. Large amounts of sediment were again found in the bottom of this junction box of this station. Figure 4-29 shows the amount of sediment in the junction box. Vertical pipe deformation for the 36-inch PVC pipe ranged from -2.8% to 0.4%. Horizontal deformation ranged from -2.1% to 0.7%. For the 36-inch HDPE pipe, the vertical deformation ranged from -3.8% to 1.4%. Horizontal deformation ranged from -4.5% to 0.6%.

**Figure 4-29: Sediment Collected in Bottom of Junction Box of Station 223+00 Pipeline**

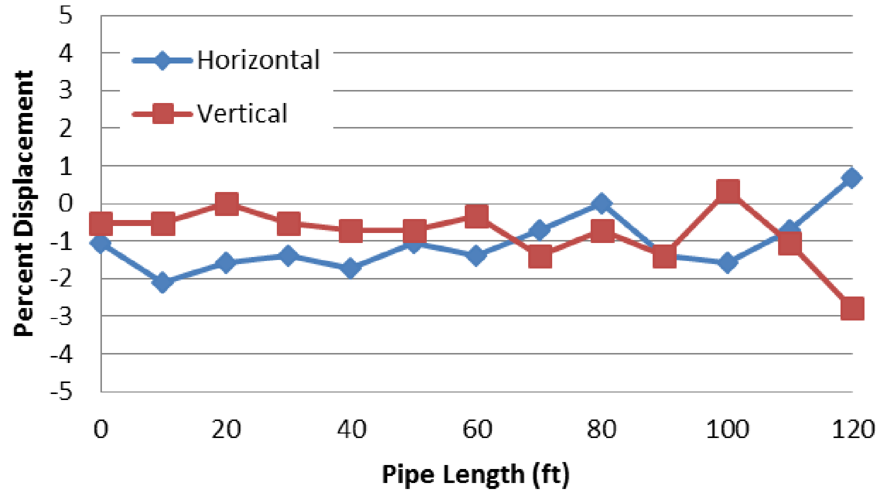



**Figure 4-30: Five Year Deflection Data for the 36-inch PVC Pipe at Station 223+00**

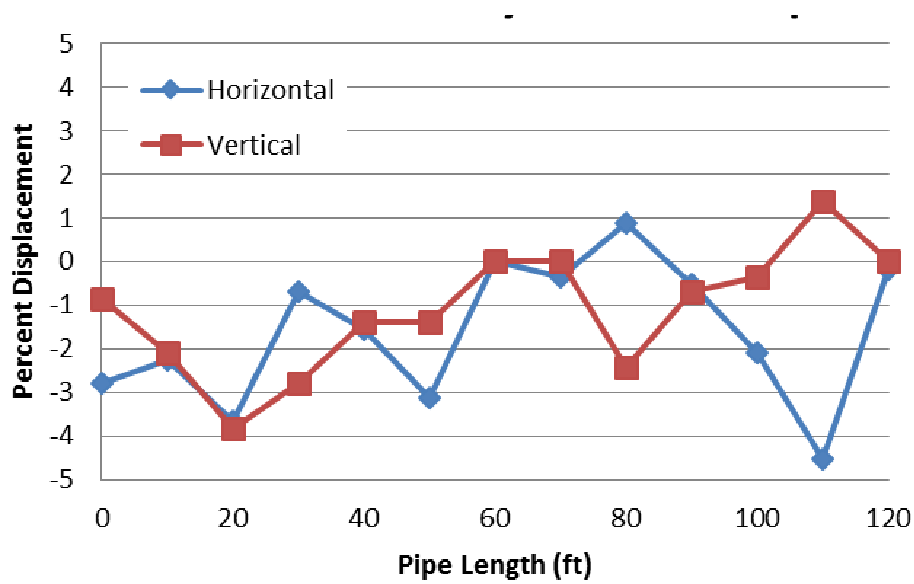



**Figure 4-31: Five Year Deflection Data for the 36-inch HDPE Pipe at Station 223+00**

*4.3.4.3 Five Year Inspection of Station 224+00*

The next station analyzed in the five year inspection was Station 224+00. The pipe ends on both the upstream and downstream ends of this station were in good condition. The concrete in both end caps was damage free and contained no chips or cracks. Signs of erosion were not significant. No separation or joint damage to the pipe joints was found. Side walls were free of any cracking, rippling, or any other damage. The junction box was filled with sediment like the two previous stations, which is illustrated in Figure 4-32. Vertical deformation in the 48-inch HDPE pipe ranged from -4.7% to -0.7%. Horizontal deformations ranged from -2.9% to 0.4%. For the 54-inch HDPE pipe the vertical deformations ranged from -5.0% to 0.3%. Horizontal deformations ranged from -1.4% to 1.0%.

**Figure 4-32: Sediment Collected in Bottom of Junction Box of Station 224+00 Pipeline**

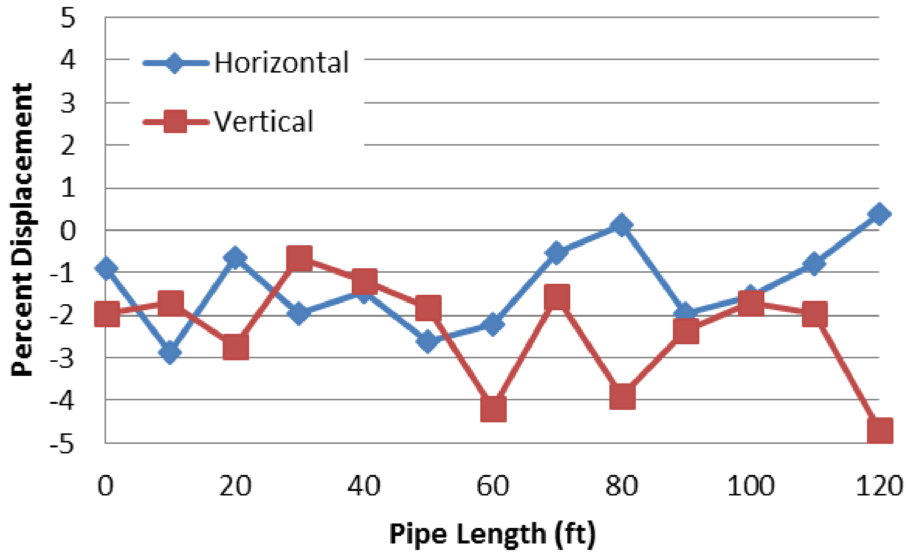



**Figure 4-33: Five Year Deflection Data for the 48-inch HDPE Pipe at Station 224+00**

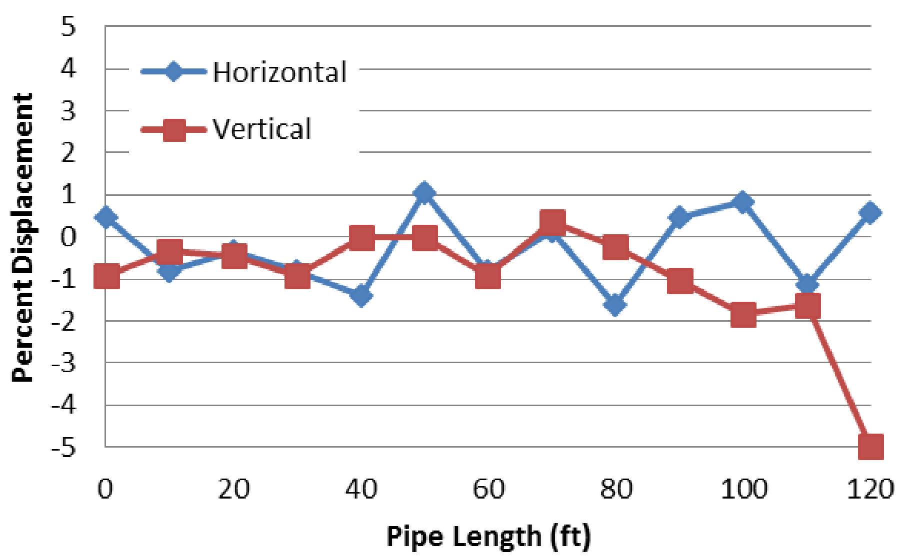



**Figure 4-34: Five Year Deflection Data for the 54-inch HDPE Pipe at Station 224+00**

*4.3.4.4 Five Year Inspection of Station 230+00*

The five year inspection for Station 230+00 started with the end caps. Each end cap remained in good condition. The concrete was free of damage and erosion was not noticed. Joints in the pipe segments were in good shape. Separations in the joints were within tolerance. Sidewalls in the pipe were in good condition. Vertical deformations ranged from -2.8% to -0.9% and horizontal deformations ranged from -2.6% to -0.5%.

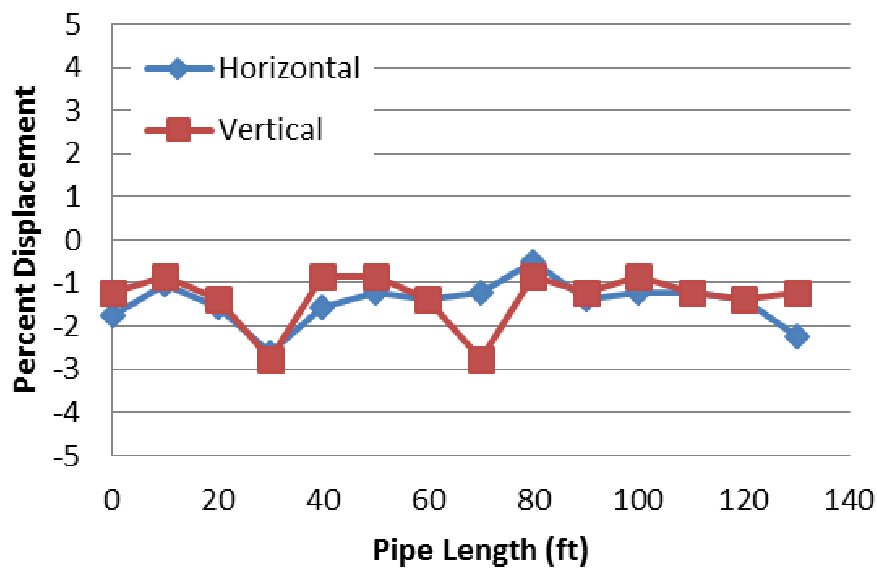



**Figure 4-35: Five Year Deflection Data for the 36-inch PVC Pipe at Station 230+00**

*4.3.4.5 Five Year Inspection of Station 231+00*

The five year inspection was completed with Station 231+00. Pipe end caps were in good condition without any chips or damage to the concrete. Pipe walls were also in good condition. No significant rippling, cracking or other damage was noticed. Pipe joints were in good condition. Vertical deformations ranged from -2.8% to 1.9%. Horizontal deformations ranged from -3.7% to 0.2%.

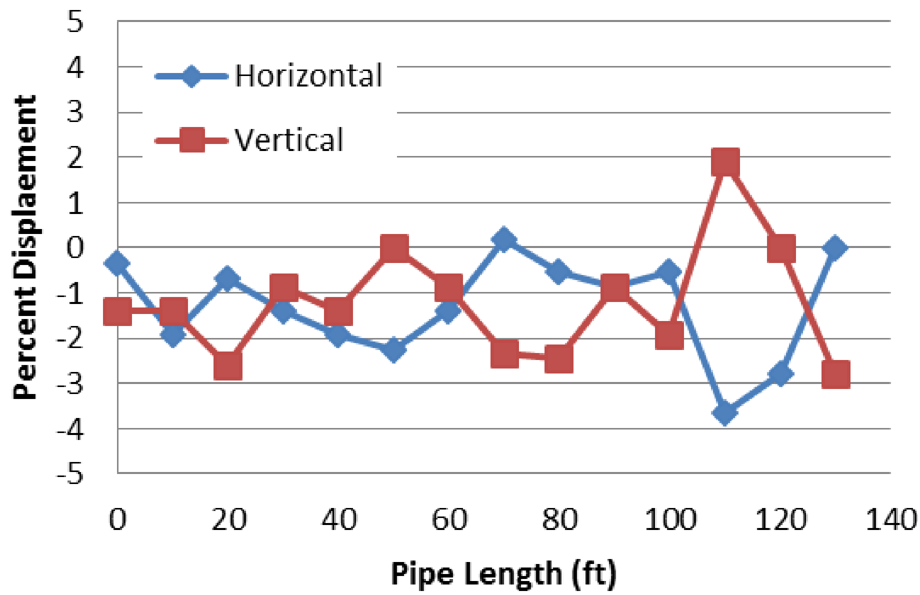



**Figure 4-36: Five Year Deflection Data for the 36-inch HDPE Pipe at Station 230+00**

### 4.4 Analysis of Stations

Since the construction and installation of the Beehive Road Site was completed, multiple inspections were completed. These inspections started soon after installation was completed and continue into 2016. The inspections included visual inspections of pipe ends, end caps, pipe walls, joints, laser deflection measurements, and four different manual deflection measurements throughout each pipeline. Using this information, the Beehive Road pipes were analyzed for their performance over these first five years.

For each pipeline, the pipe ends, end caps, and pipe walls were free of any significant damage throughout the entire study. The analysis takes into account the good condition of these components but focuses on how the deflection measurement data and other aspects will affect the long term performance of these thermoplastic pipes.

### 4.4.1 Analysis of Station 222+00

At Station 222+00 the measurements taken over the four different sessions show a similar trend (Figure 4-37 and Figure 4-38). Starting after installation in January of 2011 the PVC pipe had vertical deflections from about -2% to 1%, which are relatively small. These deflections increased steadily up to -6.4% for vertical displacement in January of 2016. The largest vertical deflections are in the center and in the segments leading to the junction box. The PVC pipe horizontal deflections begin with deflections ranging with many points between -4% and 0%. These deflections increase steadily over the subsequent five years up to 2%.

The horizontal and vertical deflection data is generally inversely related. This means that as the vertical displacement increases or decreases, the horizontal displacement decreases or increases. This follows the behavior that is expected for the pipe that as the height decreases, the width increases or vice versa. However, this is not necessarily true if the cross section shape strays from being oval. The growth in deformation over these five years of about 2% is significant. A few positions in the PVC pipe are already exceeding the 5% displacement within five years. The pattern of the deformations for thermoplastic pipes is not known, so therefore the pipes could deform relatively rapidly in the first portion of their service life and then settle and become consistent. Yet, if this deformation growth seen in the first five years continues then the pipe will continue to deflect substantially before the expected service life.

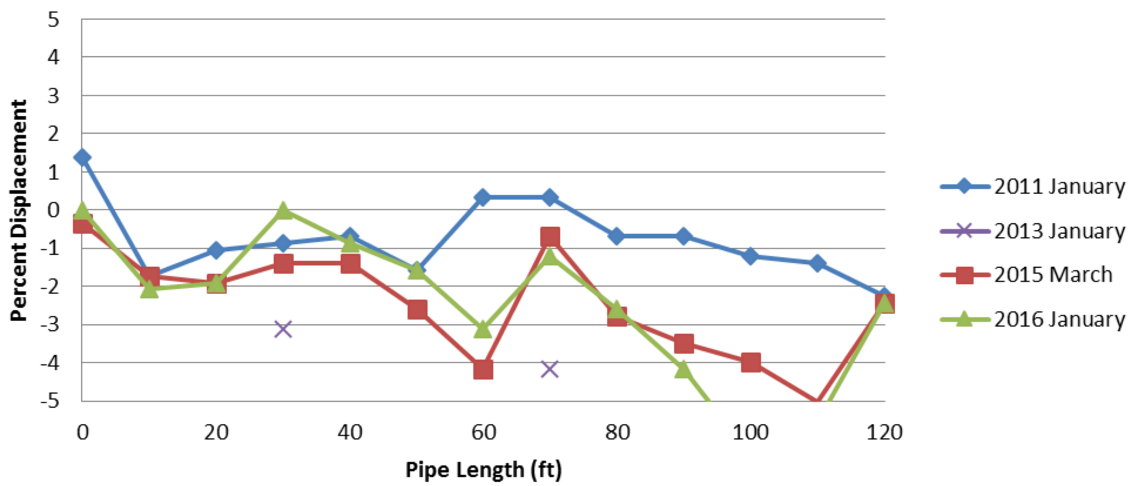



**Figure 4-37: Vertical Displacement Comparison for the 36-inch PVC Pipe at Station 222+00**

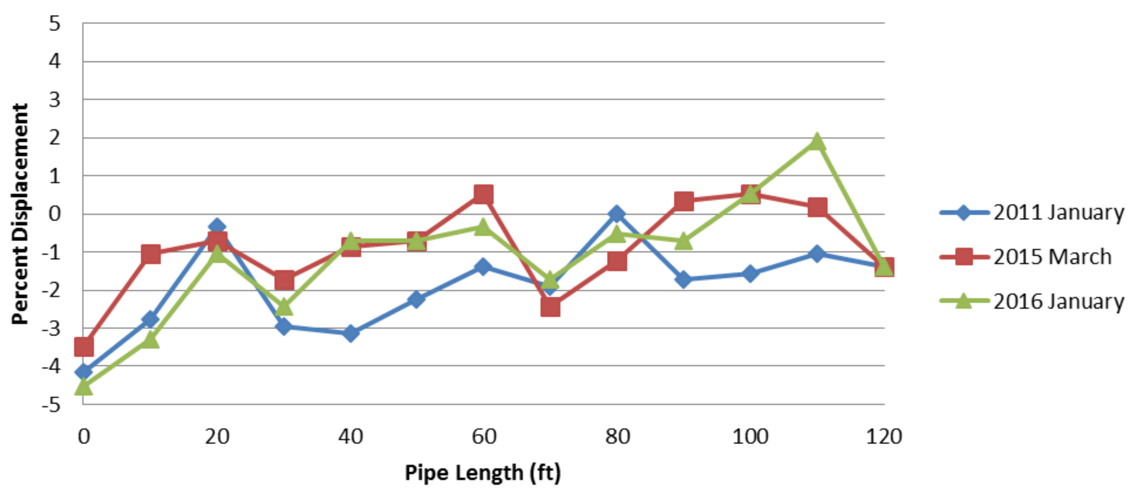



**Figure 4-38: Horizontal Displacement Comparison for the 36-inch PVC Pipe at Station 222+00**

The 36-inch HDPE pipe shows a different trend in the deflection data (Figures 4-39 and 4-40). For this HDPE pipe both the vertical and horizontal deformation is relatively constant throughout the entire line. The range in deformations then grows in both directions in the March 2015 measurements. The largest deformations are near the junction box and reach close to -5%. The data for this half of Station 222+00 is more limited than the other stations due to the presence of sediment and water, which hindered the measurement accuracy. Data was not taken during the 2016 January inspections due to sediment that was collected along the bottom of the entire pipeline.

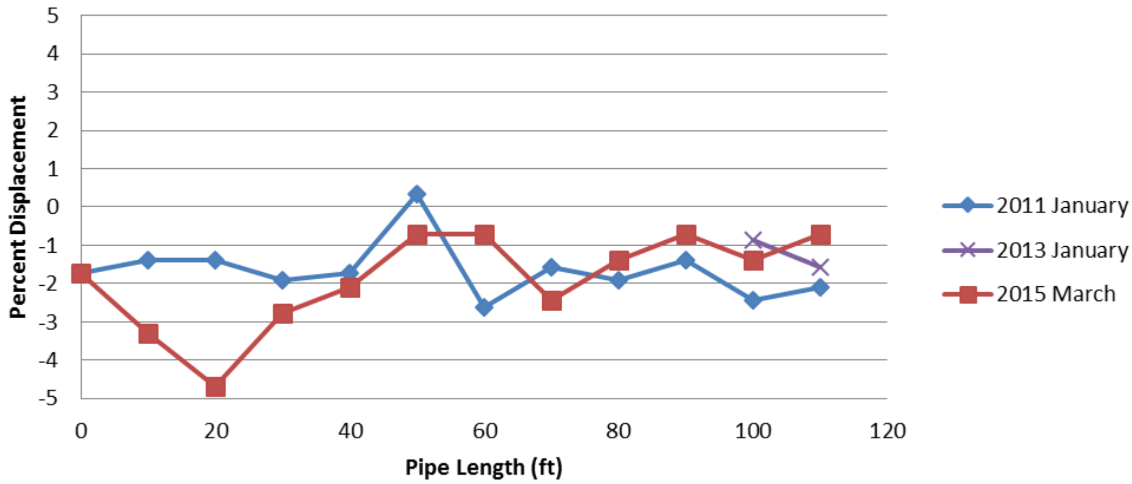



**Figure 4-39: Vertical Displacement Comparison for the 36-inch HDPE Pipe at Station 222+00**

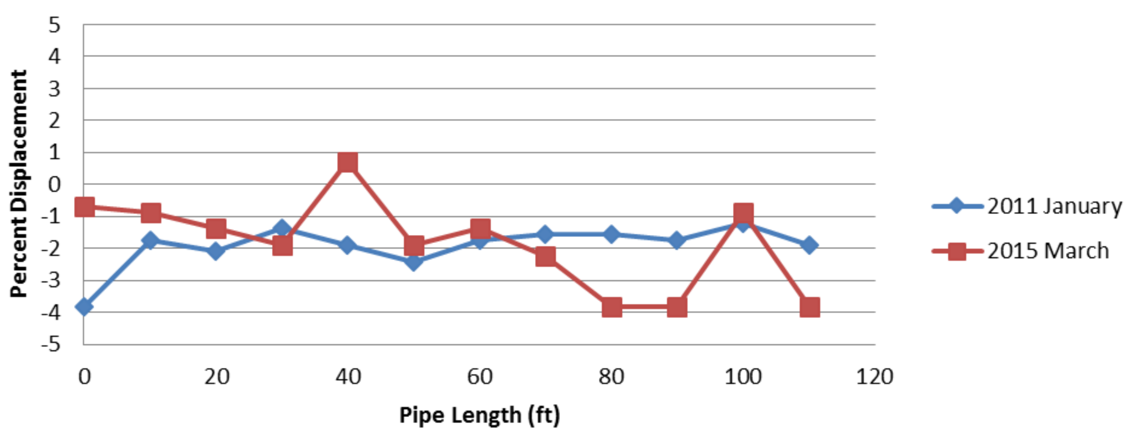



**Figure 4-40: Horizontal Displacement Comparison for the 36-inch HDPE Pipe at Station 222+00**

4.4.2 Analysis of Station 223+00

The PVC pipe at Station 223+00 shows deflection data that is different than the first station. From January 2011to March of 2015 the vertical deflections increased with a range up to -2% as can be seen in Figure 4-41. The measurements taken in January of 2016 changed against the trend of the previous three years and were in the middle between the previous measurements. The data shows that the pipes vertical height decreased steadily throughout the first three checks but then the vertical height increased again during January 2016.

The horizontal deflections followed a similar pattern throughout the multiple measurements but did not consistently increase or decrease through time. Like the vertical deflections, the January 2016 deflection measurements are actually in between the January 2011 and March 2015 deflection measurements.

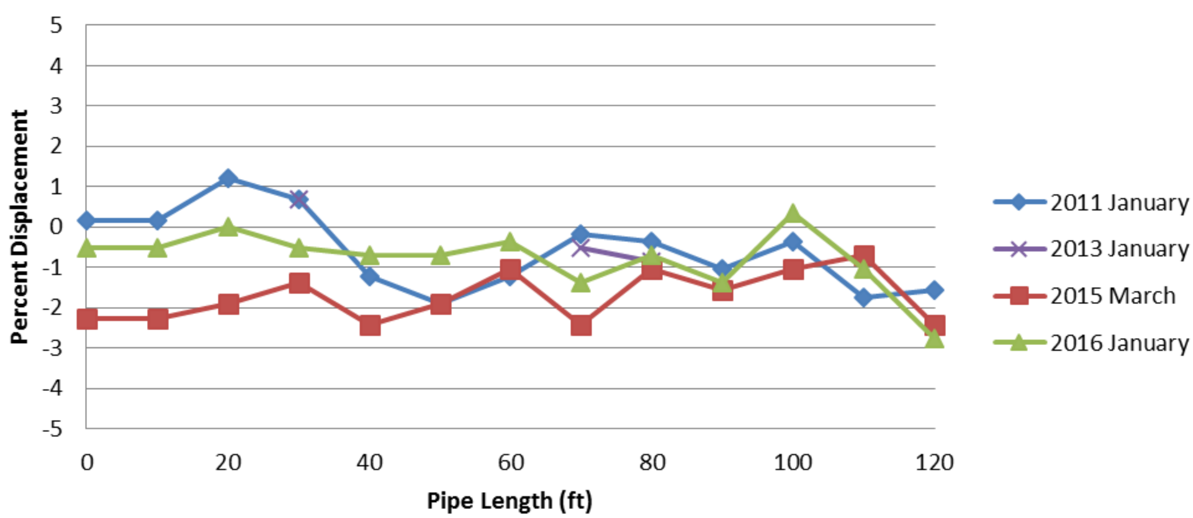



**Figure 4-41: Vertical Displacement Comparison for the 36-inch PVC Pipe at Station 223+00**

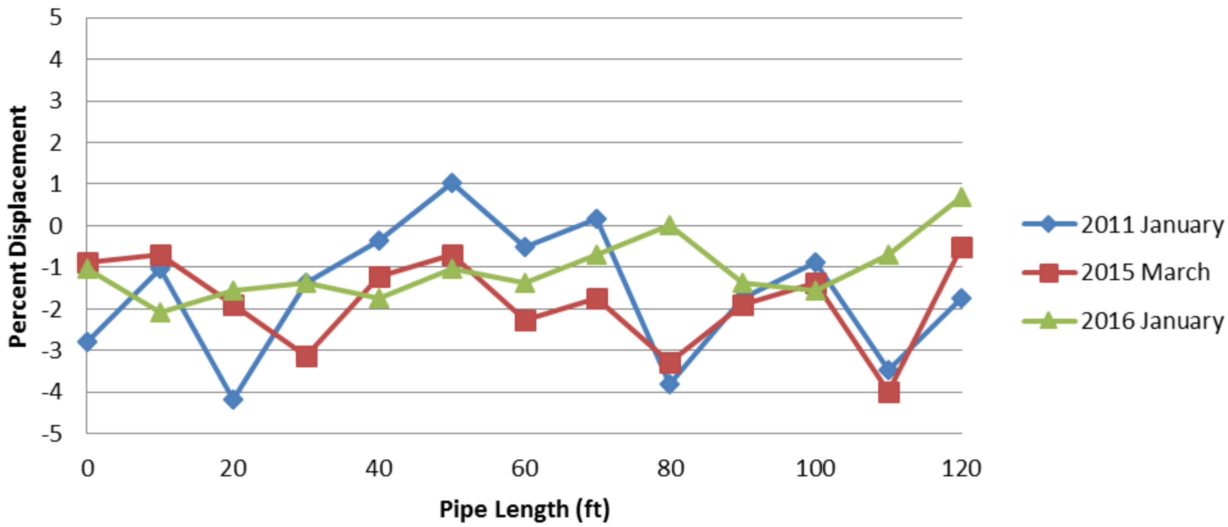



**Figure 4-42: Vertical Displacement Comparison for the 36-inch PVC Pipe at Station 223+00**

The HDPE pipe at Station 223+00 also follows a similar trend in vertical displacement as the PVC at this station. The vertical displacements increased by as much as -2% from the original measurements in January of 2011 to March of 2015. Then the displacements change in the opposite direction for the latest set of measurements where the January of 2016 deflections are actually smaller than the initial deflections. Even though the range of deflections throughout the set of measurements is not very large, the changes in direction of deflection were unexpected.

The horizontal displacements for this HDPE pipe begin at high range of negative values all the way up to -5%. For the measurements taken in 2015 and 2016, the values vary to a higher extent but have an overall lesser magnitude of measured deflection. The horizontal deflections between 2015 and 2016 are very close to each other. Each measurement made for this direction along this pipeline are almost identical and only have a difference in value of about a quarter percent.

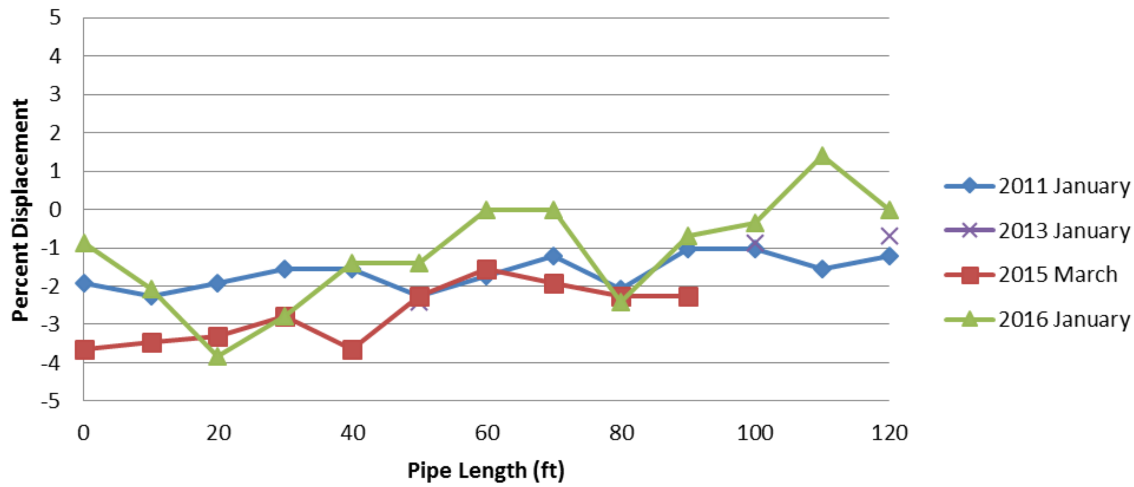



**Figure 4-43: Vertical Displacement Comparison for the 36-inch HDPE Pipe at Station 223+00**

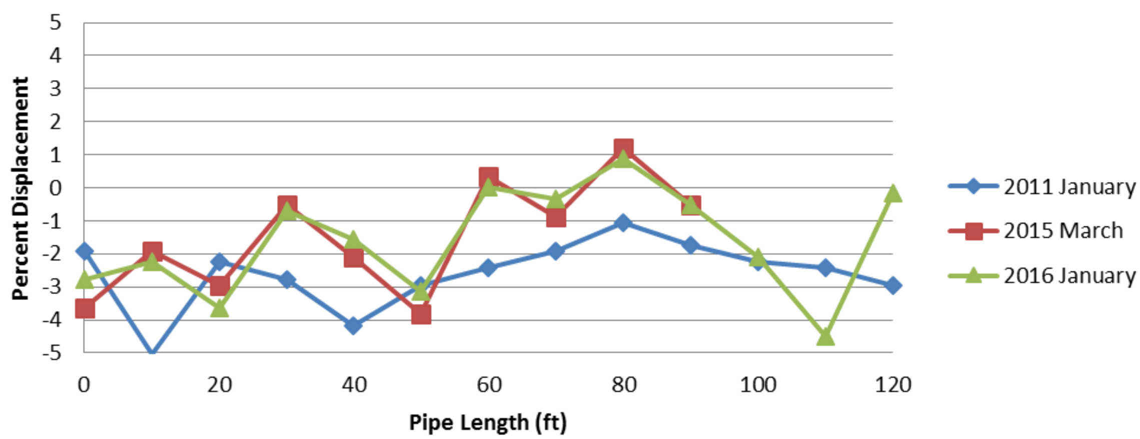



**Figure 4-44: Horizontal Displacement Comparison for the 36-inch HDPE Pipe at Station 223+00**

### 4.4.3 Analysis of Station 224+00

The pipeline at Station 224+00 involved the largest diameter pipes. The first pipeline, which was a 48-inch HDPE pipe held consistent values for vertical deflection. The horizontal deflections for the 48-inch HDPE pipe were not as consistent as the vertical deflections. The horizontal values for different points of this HDPE pipe were high in multiple places in the March 2015 readings before the deflections decreased in January 2016.

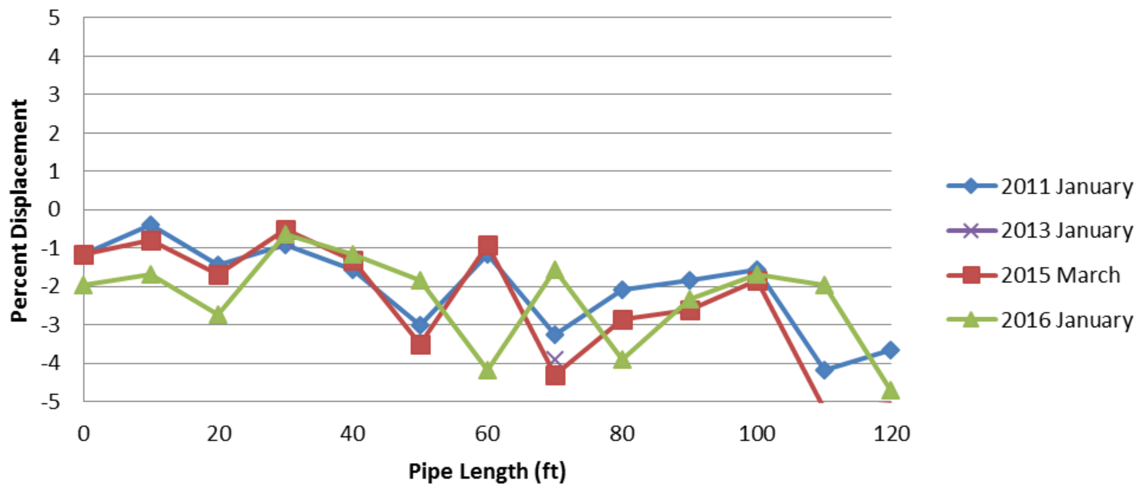



**Figure 4-45: Vertical Displacement Comparison for the 48-inch HDPE Pipe at Station 224+00**

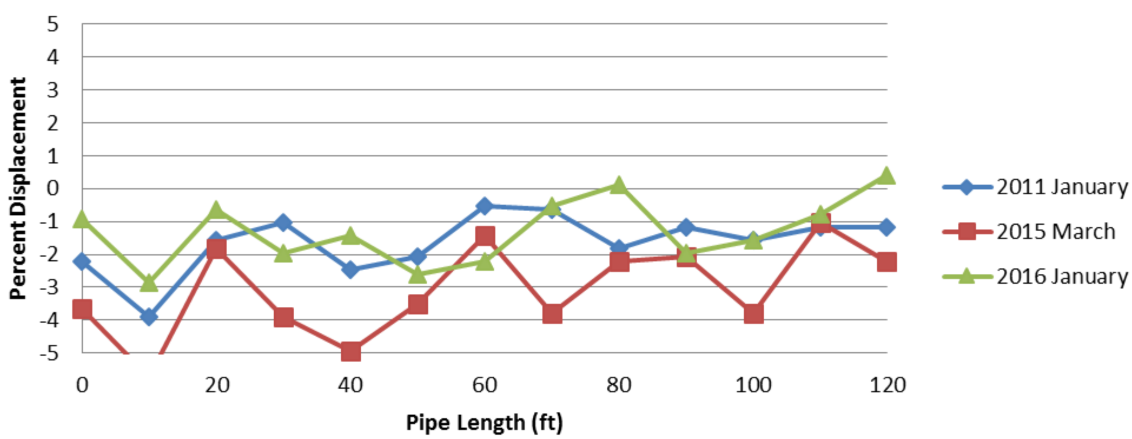



**Figure 4-46: Vertical Displacement Comparison for the 48-inch HDPE Pipe at Station 224+00**

The other half of this station contained a 54-inch HDPE pipe. It was installed under similar conditions as the 48-inch HDPE portion but resulted in different displacement patterns. This is especially evident in the horizontal deflections, which are very consistent along the entire length of the 54-inch HDPE pipe. This consistency was not observed in the horizontal deflections of the 48-inch HDPE pipe. The vertical deflections are mostly consistent throughout the pipeline but have more severe deflection changes at the two pipeline ends.

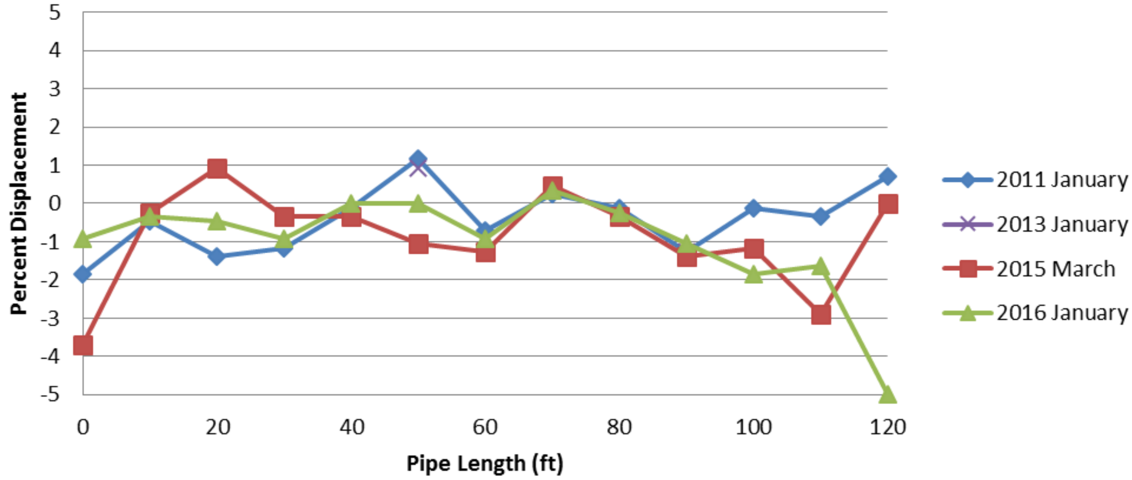



**Figure 4-47: Vertical Displacement Comparison for the 54-inch HDPE Pipe at Station 224+00**

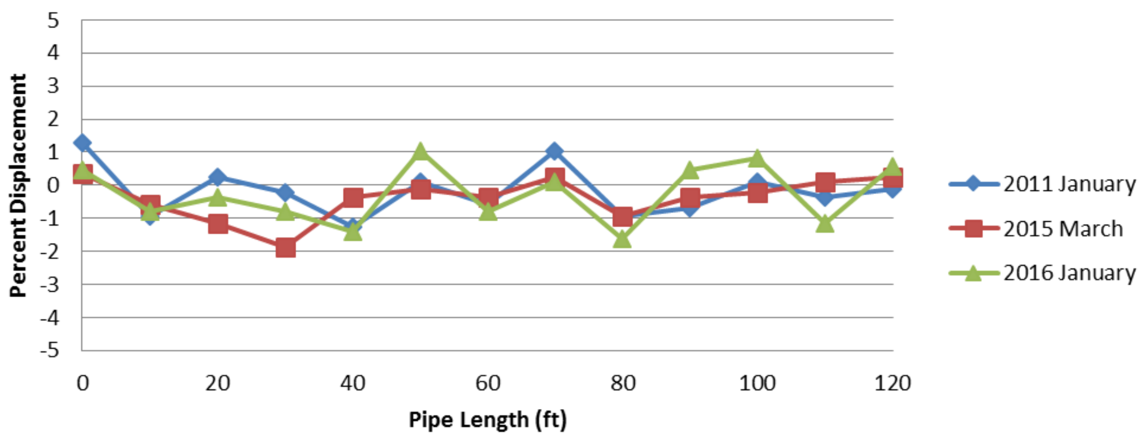



**Figure 4-48: Horizontal Displacement Comparison for the 54-inch HDPE Pipe at Station 224+00**

4.4.4 Analysis of Station 230+00

The 36-inch PVC pipe at Station 230+00 was placed under a significantly smaller cover height of about 5 to 8 feet compared to the three previous stations. The vertical displacement measurements for this pipe showed some of the first positive displacements, meaning the pipe diameter actually increased vertically compared to the original diameter in some locations. Overall, the vertical deflections for this pipeline are not very large and have a maximum value of about -3%. The horizontal deflections are consistent over the years of measurement at about -1% or -2%.

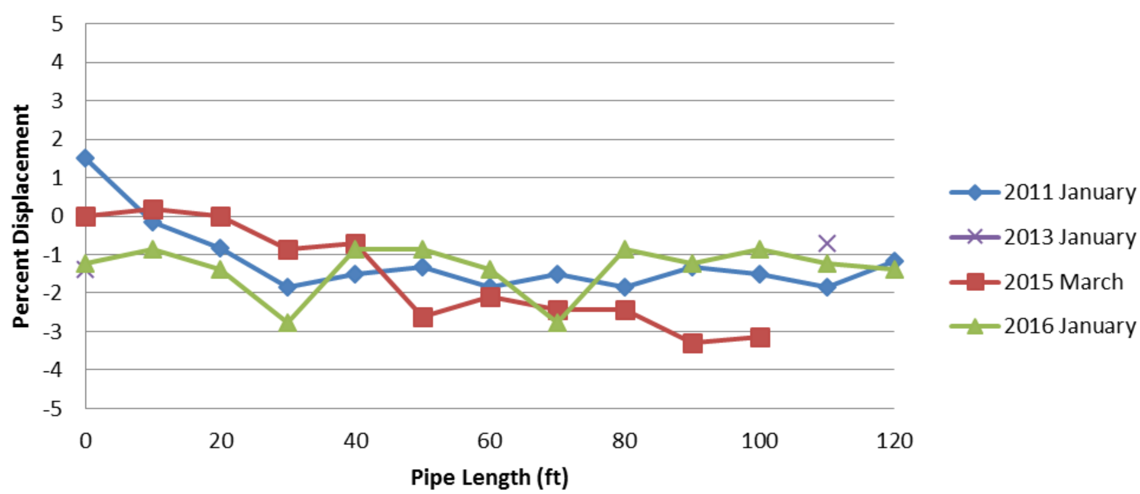



**Figure 4-49: Vertical Displacement Comparison for the 36-inch PVC Pipe at Station 230+00**

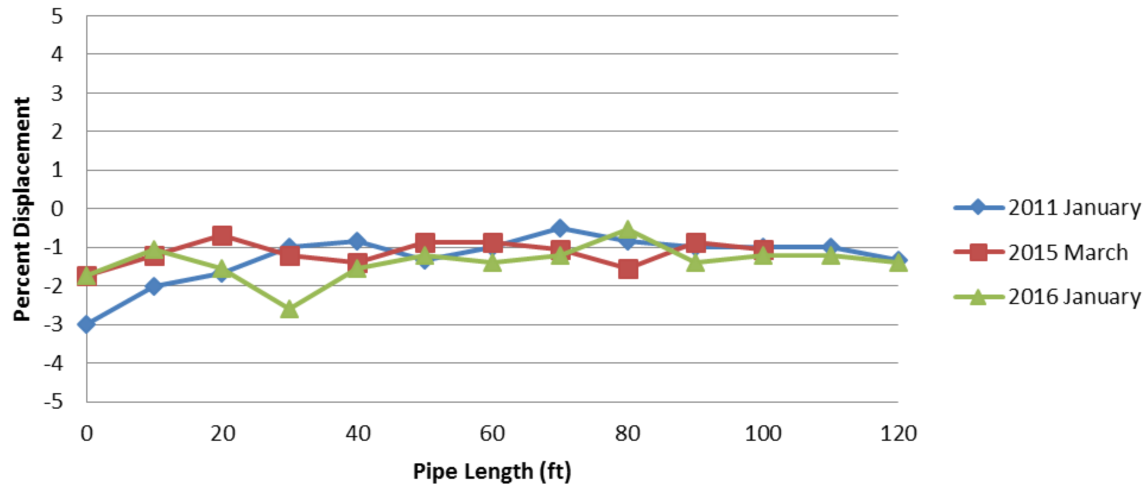



**Figure 4-50: Horizontal Displacement Comparison for the 36-inch PVC Pipe at Station 230+00**

4.4.5 Analysis Station 231+00

Station 231+00 consists of a 36-inch HDPE pipe that was also under a smaller cover height of about 5 to 8 feet. The horizontal displacement was consistent through the years other than significant change in deflections toward one end of the pipeline. Vertical deformations varied throughout the length but magnitudes were relatively small.

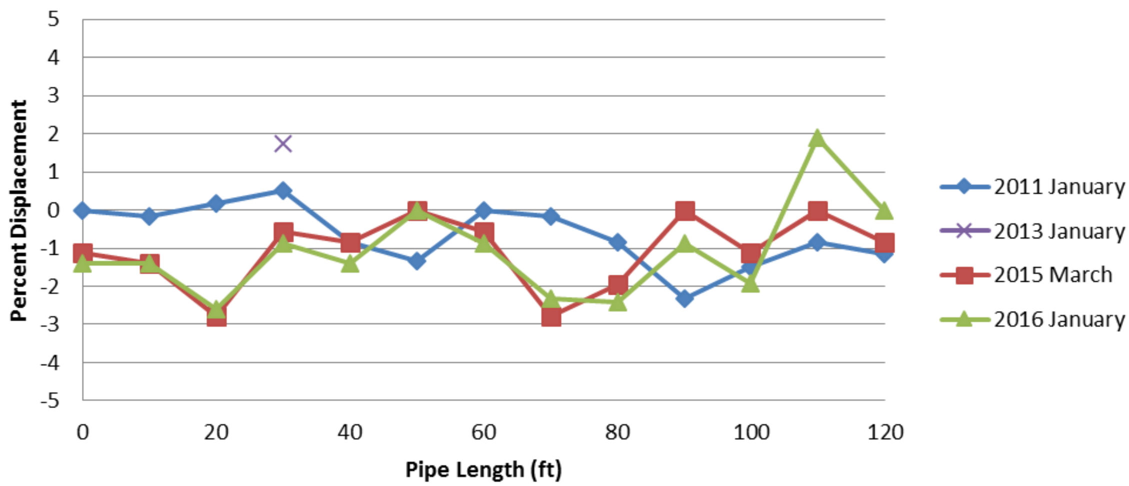



**Figure 4-51: Vertical Displacement Comparison for the 36-inch HDPE Pipe at Station 231+00**

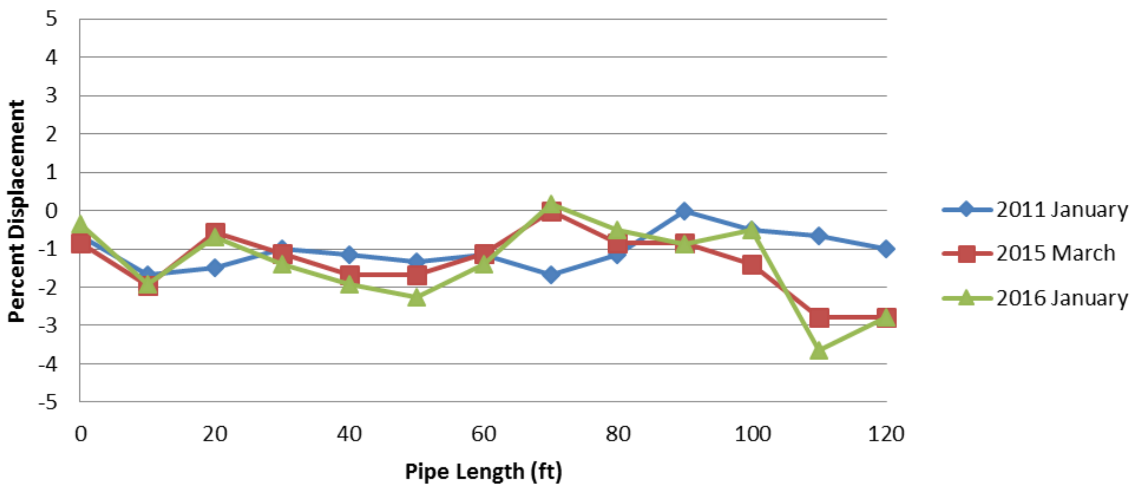



**Figure 4-52: Horizontal Displacement Comparison for the 36-inch HDPE Pipe at Station 231+00**

### 4.4.6 Sediment in Junction Boxes

At the fifth year inspection, large amounts of sediment were found in the junction boxes, even though these pipelines were built for research purposes and were not supposed to service any water at any time. It is believed that this sediment was carried into the pipes from the uphill ends as rain water running down the slope of the hill entered the pipe end and traveled, carrying the soil, through the pipeline. Because the pipes entering the junction box are inserted toward the middle of each side of the box, any water or sediment traveling through the pipe is collected in the bottom section of the box. Before the five year inspection, the junction boxes were relatively sediment and water free. The sediment found in all three of the junction boxes probably resulted from large storms that occurred in the fifth year timeframe.

### 4.4.7 Pipe Materials

The Beehive Road Site compared two different thermoplastic piping materials - PVC and HDPE. The data collected at the site for these two materials shows how each performs under the different culvert situations. So far, the PVC and HDPE performance does not differ significantly. Both pipe materials had similar conditions in pipe walls and pipe joints. There was no significant difference in deformation data for the two pipe materials. Both materials followed a similar condition and deformation for the other differing variables such as cover height. Although evaluating these two plastic materials is important, especially from a long-term performance standpoint, it is equally important to recognize that material stiffness (modulus of elasticity) is only a portion of the total rigidity (modulus of elasticity time effective moment of inertia). The other contributor to the total rigidity of a given pipe product is the design of the wall geometry. In other words, a pipe product made of a less stiff material may provide a greater rigidity if the effective moment of inertia is greater.

### 4.4.8 Backfill Material

Three different classes of soils (Class I, Class II, and Class III) were used for cover over the pipelines. After analyzing the data for the stations based on the different classes of soils, definitive conclusions cannot be made at this time. Conditions in the pipe walls and pipe joints were all similar. There are no identifiable differences in the pipe deformations data between the different classes of soil used. Each type appears to produce similar deformation results at this five year mark.

### 4.4.9 Cover Height

Two different cover heights were used for the Beehive Road Site. Three of the Stations, (Station 222+00, Station 223+00, and Station 224+00) used a cover height of about 25 to 30 feet. The other two Stations (Station 230+00 and Station 231+00) used a cover height of 4.5 to 8 feet. The deeper cover is intended to evaluate the performance of those pipes under significant soil burden pressure whereas the shallow-buried pipes were subjected to traffic loads. The ADT of the roadway and the extent of heavy truck traffic however is not known at this time. Every pipe at each station had generally the same condition in terms of end cap, pipe joint, and pipe wall condition. There was a difference in the condition for the deflection data for the two differing cover heights. The pipelines at Station 230+00 and Station 231+00 had smaller deflections. The variations in deflections through the length of the pipes at Stations 230+00 and 231+00 were also significantly less. The deflection changes were smooth and did not change abruptly as much as deflections in Station 222+00, Station 223+00, and Station 224+00 where the cover height was higher. The higher cover stations reached and passed 5% deflection in several locations. Deflection at the smaller cover stations remained under 3% in either direction except for one point where it reached -3.7%. Overall, the data suggests that the pipelines deflections are significantly smaller under smaller cover height.

### 4.4.10 Pipe Diameter

Three different pipe diameters were used throughout the Beehive Road Site. The majority of the pipes had a diameter of 36 inches, but 48 inches and 54 inches were also used in the diameters of one pipeline. The pipe wall conditions and pipe joints were similar for all diameters. Because 6 of the 8 different pipelines were 36 inches in diameter, one was 48 inches in diameter, and one was 54 inches, there is limited data to draw conclusions for the performance of thermoplastic pipes based on different diameters. From the data collected at Beehive Road, no differences or trends can be discerned for the different pipelines based on the diameter sizes.

#### 4.4.11 Overall Analysis

The data and observations from the Beehive Road site show that the pipes have performed satisfactorily up to this five year benchmark.  In the five years since the pipes were installed, each pipeline has showed differing results. Even though no detrimental damage, cracking, or rippling is found in the pipelines, the variation in deflections throughout the pipelines may be significant given that they are only five years into their service life. The year-to-year condition changes over the first five years demonstrates that their performance can be non-uniform and unpredictable. As the pipe is in use over 80+ years, the chances that these variable changes will increase and possibly result in failure during its service life is high.

### 4.5 Laser Scan Inspection

When the original Beehive Road installation was completed, a laser scan was conducted along the entire length of each pipe. This scan provided comprehensive diameter data and videography for every section of the pipe. The full results of that scan are presented in Shepard et al. (2011).  Additional laser scans taken approximately every five years after the original can be used to conduct further assessment and comparison.

Collecting laser scans at these intervals will be of significant benefit for the overall monitoring of long-term performance. These findings will allow ALDOT to determine the potential benefits conducting laser scans of other pipe installations on highway projects in the state.

As part of this Phase 2 project, another laser deflection scan was conducted in July of 2016. The research team thoroughly investigated companies that routinely perform buried pipe laser scans and chose AET Services out of Liberty South Carolina. The research team’s experience with AET was very good. The scan of all pipes was completed in approximately six hours and the report and videos delivered approximately ten days later. Images of the scan procedure are shown in Figures 53 through 58.

The full inspection report is provided in Appendix A; the laser inspection videos were also delivered to ALDOT. Overall, the laser inspection corroborated the direct measurements made by the team that identified only a few isolated locations that exceeded the 5% limit. The AET report pointed to a potential instability within Station 222 (36-inch HDPE), significant deformation at the junction box at Station 222 (36-inch PVC), and significant deformation at one location within Station 224 (54-inch HDPE).

**Figure 4-53: Preparing Laser Robot for Pipe Inspection**

**Figure 4-54: Laser Robot Entering PVC Pipe**

**Figure 4-55: Top View of Laser Robot Entering HDPE Pipe**


**Figure 4-56: Data Collection Roadside**

**Figure 4-57: Data Collection Valley**


**Figure 4-58: Data Collection Close-up**

## 4.6 Beehive Road Field Monitoring Summary

The Beehive Road site was constructed in 2011 in order to examine the performance of PVC and HDPE pipes as cross-drain culverts. Over the five years of service since it was initially installed, the pipelines have given researchers an understanding of the initial performance of these thermoplastic pipes under differing cross-drain scenarios. No significant condition issues in terms of the conditions of end caps, pipe walls, and pipe joints have occurred in any pipeline. However, deflection data differs for each pipeline scenario. Specific trends for causation for these differing states or deflections cannot be defined at this time. The numerous changes in the conditions of each pipeline over these first five years shows the unpredictable nature of these thermoplastic pipes. It also indicates that the use of thermoplastic pipes as cross-drains should likely be limited to certain conditions or situations in order to successfully achieve the intended service life.

# SECTION 5

# PIPE SELECTION DECISION ALGORITHM BACKGROUND

## 5.1 Background

The primary objective of the decision algorithm task was to develop a practical culvert pipe selection approach that can be used by state and county highway engineers. It focuses on circular cross section manufactured pipes that serve up to 60 inches in diameter; cast-in-place and segmental products are not considered. It is also limited to trench installations that are typical of highway cross drain applications. The algorithm helps to determine the optimum type and class of pipe for cross-drainage application. The objective was achieved by integrating all of the research findings from the Phases 1 and 2 Plastic Pipe for Highway Construction Project and critically analyzing the material properties, serviceability, durability, and installation requirements, as well as case studies and reports for the following culvert materials:

- Class II, III, IV, and V Precast Reinforced Concrete
- Plastic (corrugated high density polyethylene, corrugated polyvinyl chloride, corrugated polypropylene)
- Corrugated Aluminum
- Corrugated Aluminized Steel
- Corrugated Galvanized Steel

This analysis laid the groundwork for weighing the considerations between plastic pipe, concrete pipe, and metal pipe for a specific highway construction project. *The Specification for Culvert Material Selection* was developed from the conclusions reached by this comparison and by the initial input parameters defined in *Cross-Drain Pipe Material Selection Algorithm.* The purpose of *The Specification for Culvert Material Selection* was not only to assist highway engineers with choosing the optimum solution but to help make and defend contract decisions. *The Specification for Culvert Material Selection* may be found in Appendix B.

### 5.2 Typical Culvert Shapes

Culverts are available in a wide range of shapes and configurations. The most common culvert shape is circular, but other typical shapes include pipe arch, elliptical, box, frame, and multiple barrel. Culvert selection factors include roadway profiles, channel characteristics, flood damage elevations, construction and maintenance costs, and estimates of service life (Schall et al. 2012).

As described in the Wisconsin Department of Transportation (WisDOT) *Structure Inspection Manual*, typical culvert shapes are defined below (WisDOT 2011).

- Circular – This is the most common culvert shape. Although hydraulically and structurally efficient, circular culverts can reduce a stream's width and circular culverts are more prone to clogging than any other shape.

- Pipe Arch – The pipe arch culvert is used when the distance from the stream bottom to the roadway is limited. The culvert is arched on top and flattened on the bottom. Pipe arch culverts are prone to clogging.

- Elliptical – Elliptical culverts have the same advantages and disadvantages as pipe arch culverts.

- Box – Box culverts are adaptable for many site conditions. Square or rectangular in shape, box culverts always have a floor ensuring the natural stream bed is covered.

- Frame – Similar to box culverts, but frame culverts do not have a floor allowing the natural stream bed to be exposed.

- Multiple Barrel – Multiple barrel or cell culverts are a series of pipes, arches, or boxes placed side by side. Multiple barrel culverts are commonly used when the distance from the stream bottom to the roadway is limited. The major disadvantage to using multiple barrel culverts is that waterway debris is easily snagged by the cell walls or soil between the openings.

### 5.3 Culvert Materials

Section 12 of the American Association of State Highway and Transportation Official's (AASHTO) *Load and Resistance Factor Design* (*LRFD) Bridge Design Specifications* identifies four materials approved for use as circular pipe for buried structures (AASHTO 2009).

- Aluminum Pipe (ASTM B745)
- Precast Concrete Pipe (ASTM C76)
- Steel Pipe (ASTM A760)
- Thermoplastic Pipe (AASHTO M294)

In 2014, The National Cooperative Highway Research Program (NCHRP) surveyed transportation agencies across North America. One of the questions asked was, "Which of the following pipe material types does your agency currently use or is considering for use?" Based on the responses shown in Figure 5-1, concrete is the most commonly used culvert material in North America with galvanized steel and high density polyethylene following closely behind.

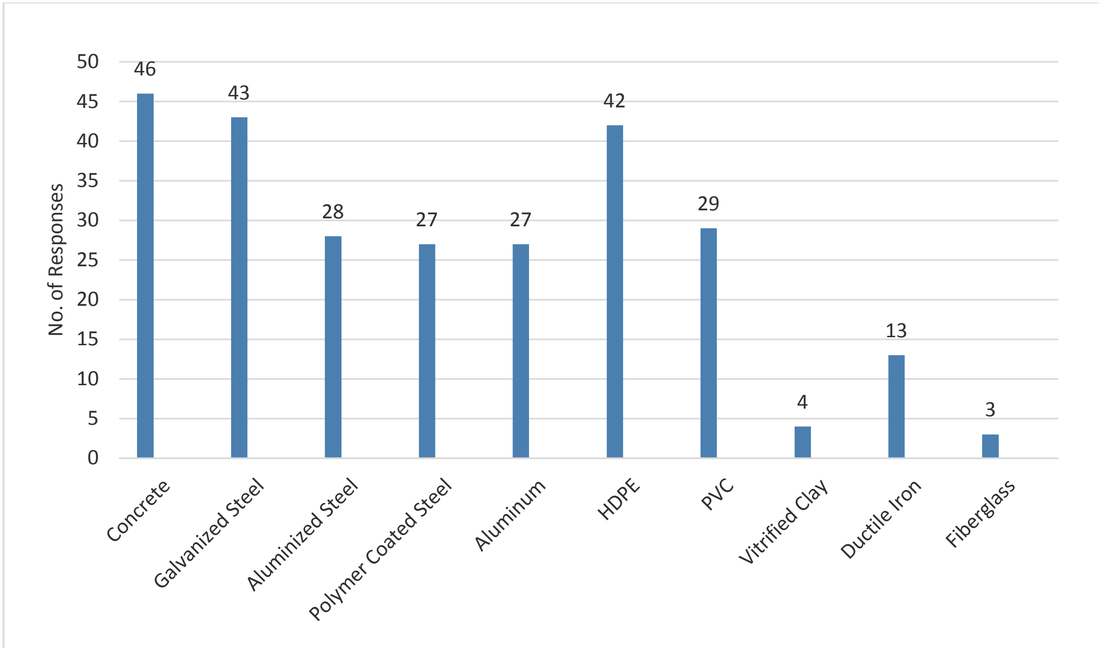



**Figure 5-1: Pipe Material Types in Use or Being Considered for Use (Maher et al. 2015)**

### 5.3.1 Reinforced Concrete

According to the American Concrete Pipe Association (ACPA), concrete pipe has been used in the United States for over a century. The article, "Century Concrete Pipe Does Exist" written by A. Grant Lee of the Canadian Concrete Pipe Association, claims that the earliest recorded use of concrete pipe in the United States was constructed between 1840 and 1842 in Mohawk, New York, at the home of General Francis Elias Spinner. The concrete pipes were used to convey domestic sewage to the Erie Canal.

> *In 1982 (140 years after installation), the pipeline was exhumed and found to be in excellent condition, and still functioning. Details about America's earliest sewers are rare, but it is known that concrete pipe was used for sanitary sewers to control outbreaks of Yellow Fever in the mid-1800s.* (Lee n.d.)

It was not until 1867 that the idea for reinforced concrete was patented by a French gardener named Joseph Monier. Frustrated with the brittleness of clay, Monier began strengthening his cement flower pots with wire mesh. Figure 5-2 illustrates Monier's initial design for a reinforced flower pot. He patented the idea in 1867 and debuted his invention that same year at the Paris Exposition. According to Britannica, Monier extended the application to other engineering structures, such as railway ties (sleepers), pipes, floors, arches, and bridges. (Encyclopedia Britannica n.d.)

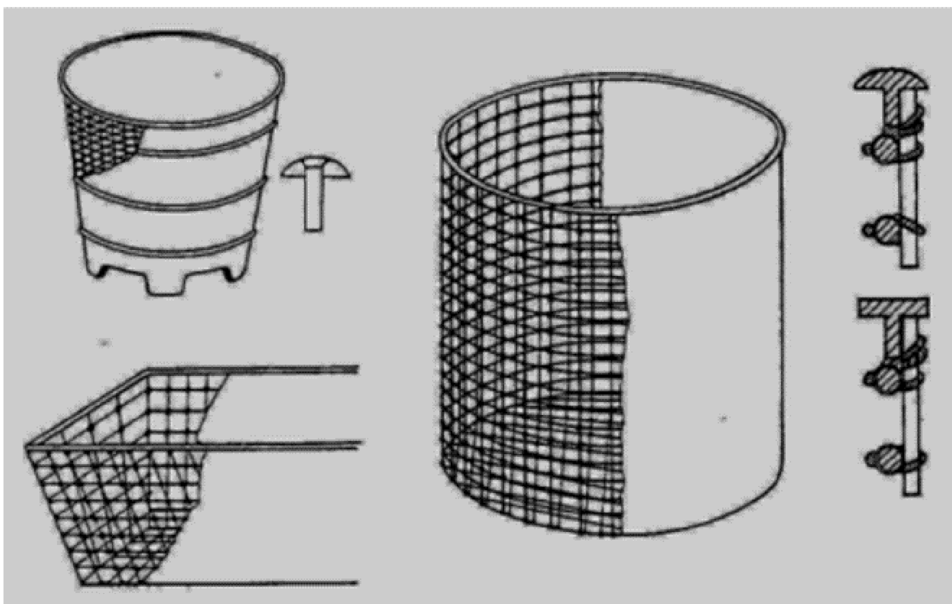


**Figure 5-2: Monier's Design for a Reinforced Flower Pot (Barbisan and Guardini 2005)**

The basic materials that are used to create concrete are Portland cement, aggregates, and water. As the Portland cement mixes with water, a paste begins to form that coats the aggregates. The paste then hardens and gains strength through a chemical reaction known as hydration. This paste will eventually form concrete. However, the process is not as simple as mixing several materials together. Improper proportioning can lead to an inferior product. According to the Portland Cement Association's (PCA) article, "How Concrete is Made",

> *A mixture that does not have enough paste to fill all the voids between the aggregates will be difficult to place and will produce rough surfaces and porous concrete. A mixture with an excess of cement paste will be easy to place and will produce a smooth surface; however, the resulting concrete is not cost-effective and can more easily crack.* (PCA n.d. how-concrete-is-made)

There are five basic methods for producing concrete pipe, which are differentiated based on the concrete mix. Wet casting is the most common method used to manufacture large diameter pipe. Wet casting uses a high-slump concrete mix with a slump typically less than four inches. The other four methods include: centrifugal/ spinning, dry cast, packerhead, and tamp-entail.

Concrete may be precast or cast-in-place. Precast concrete elements are manufactured in a plant then transported to the construction site. Cast-in-place concrete structures are constructed at the construction site. Precast concrete is typically more efficient and timesaving because weather will not delay the manufacturing process. "Precast concrete is more popular for smaller, cross-drain pipe applications because it can be manufactured in a controlled environment and save significant installation time" (PCA n.d. concrete-pipe).

Reinforced concrete culverts must meet the requirements of American Society for Testing and Materials (ASTM) C76 (or AASHTO M170) *Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe*. Concrete pipe is specified according to strength class and inside diameter. The five strength classes are identified as

Class I, Class II, Class III, Class IV, and Class V and correspond to varying D-loads. This correlation is shown in Table 5-1.

**Table 5-1: Required D-Load Capacity per Strength Class (ASTM C76)**

| Class | 0.01-Inch Crack D-Load | Ultimate D-Load |
|---|---|---|
| | (lbs/feet/feet) | (lbs/feet/feet) |
| I | 800 | 1200 |
| II | 1000 | 1500 |
| III | 1350 | 2000 |
| IV | 2000 | 3000 |
| V | 3000 | 3750 |

Reinforced concrete culverts must also meet the requirements of ASTM C655 *Standard Specification for Reinforced Concrete D-Load Culvert, Storm Drain, and Sewer Pipe* when the culvert is designed for specific D-loads. According to ASTM C655, strength of a reinforced concrete culvert is designated as follows:

> *The design strength designation of the pipe shall be the D-load to produce the 0.01-in. crack. The relationship of ultimate strength D-load to the design strength D-load shall be determined using a factor of 1.5 for design strength designations up to 2000 lbf/ft·ft of diameter, a factor varying in linear proportions from 1.5 to 1.25 for design strength designations from 2000 through 3000, and a factor of 1.25 for design strength designations in excess of 3000*. (ASTM C655)

**Table 5-2: Standard Designated Inside Diameter, inch (ASTM C655)**

| | | | | | | |
|---|---|---|---|---|---|---|
| 12 | 24 | 36 | 60 | 84 | 108 | 132 |
| 15 | 27 | 42 | 66 | 90 | 114 | 138 |
| 18 | 30 | 48 | 72 | 96 | 120 | 144 |
| 21 | 33 | 54 | 78 | 102 | 126 | |

### 5.3.2 Corrugated Steel

In 1895, two residents of Indiana, E. Stanley Simpson and James Watson, sent a patent application to Washington, D.C. for their invention of a corrugated metal culvert. Watson wanted to create a corrugated sheet-metal pipe to replace the existing vitrified tile that was currently being used as a culvert material. Vitrified tile was very heavy, which required the material to be manufactured in short lengths for ease of transportation. As horse drawn wagons were the sole transportation of that time, vitrified tile culverts were never manufactured longer than 20 feet.

As stated in the article "There are Interesting Inventions," the two inventors had difficulty finding investors despite the widespread consensus that the product was strong, durable and had a long life expectancy. Several of the engineers in Crawfordsville doubted the new invention so strength tests were performed. "Steam tractors and even an elephant from a visiting circus tested the strength of the Simpson-Watson invention" (Baldwin 2014). The installation of Simpson's and Watson's culvert is shown in Figure 5-3.

**Figure 5-3: Installation of Corrugated Metal Culvert (Baldwin 2014)**

Corrugated steel culverts must meet the requirements of ASTM A760 (or AASHTO M36) *Standard Specification for Corrugated Steel Pipe, Metallic-Coated, for Sewers and Drains*. Corrugated steel pipe is classified based on the pipe's cross-sectional geometry and type of corrugations. An example of corrugated steel pipe sizes is shown in Table 5-3. The classifications considered for this project include: Type I, Type IA, Type IR, and Type IS. According to ASTM A760, Type I, Type IA, Type IR, and Type IS are applicable to pipe having a full circular cross section.

Type I is fabricated with annular (circumferential) or helical corrugations with a single thickness of corrugated sheet. Type IA is fabricated with helical corrugations and lock seams with an outer shell of corrugated sheet and an inner liner of smooth sheet. Type IR is fabricated with helical ribs projecting outwardly with a single thickness of smooth sheet. Type IS is fabricated with helical ribs projecting outwardly with metallic coated steel inserts.

## Table 5-3: Corrugated Steel Pipe Sizes (ASTM A760)

| Nominal Inside Diameter | | Corrugation Sizes[A] | | | | Ribbed Pipe | | | Minimum Outside Circumference[B] | |
|---|---|---|---|---|---|---|---|---|---|---|
| in. | mm | 1½ by ¼ in. [38 by 6.5 mm] | 2⅔ by ½ in. [68 by 13 mm] | 3 by 1 in. [75 by 25 mm] | 5 by 1 in. [125 by 25 mm] | ¾ by ¾ by 7½ in. [19 by 19 by 190 mm] | ¾ by 1 by 11½ in. [19 by 25 by 292 mm] | ¾ by 1 by 8½ in. [19 by 25 by 216 mm] | in. | mm |
| 4 | 100 | X | | | | | | | 11.4 | 264 |
| 6 | 150 | X | | | | | | | 17.7 | 441 |
| 8 | 200 | X | | | | | | | 24.0 | 598 |
| 10 | 250 | X | | | | | | | 30.2 | 755 |
| 12 | 300 | X | X | | | | | | 36.5 | 912 |
| 15 | 375 | X | X | | | X[C] | | | 46.0 | 1148 |
| 18 | 450 | X | X | | | X | X | X | 55.4 | 1383 |
| 21 | 500 | | X | | | X | X | X | 64.8 | 1620 |
| 24 | 600 | | X | | | X | X | X | 74.2 | 1854 |
| 27 | 675 | | X | | | X | X | X | 83.6 | 2091 |
| 30 | 750 | | X | | | X | X | X | 93.1 | 2483 |
| 33 | 825 | | X | | | X | X | X | 102.5 | 2561 |
| 36 | 900 | | X | X | X | X | X | X | 111.9 | 2797 |
| 42 | 1050 | | X | X | X | X | X | X | 130.8 | 3269 |
| 48 | 1200 | | X | X | X | X | X | X | 149.6 | 3739 |
| 54 | 1350 | | X | X | X | X | X | X | 168.4 | 4209 |
| 60 | 1500 | | X | X | X | X | X | X | 187.0 | 4675 |
| 66 | 1650 | | X | X | X | X | X | X | 205.7 | 5142 |
| 72 | 1800 | | X | X | X | X | X | X | 224.3 | 5609 |
| 78 | 1950 | | X | X | X | X | X | X | 243.0 | 6075 |
| 84 | 2100 | | X | X | X | X | X | X | 261.7 | 6542 |
| 90 | 2250 | | | X | X | X | X | X | 280.3 | 7008 |
| 96 | 2400 | | | X | X | X | X | X | 299.0 | 7475 |
| 102 | 2550 | | | X | X | X | X | X | 317.6 | 7941 |
| 108 | 2700 | | | X | X | X | X | X | 336.3 | 8408 |
| 114 | 2850 | | | X | X | X | | X | 355.0 | 8874 |
| 120 | 3000 | | | X | X | X | | X | 373.6 | 9341 |
| 126 | 3150 | | | X | X | | | X | 392.3 | 9807 |
| 132 | 3300 | | | X | X | | | X | 410.9 | 10274 |
| 138 | 3450 | | | X | X | | | X | 429.6 | 10740 |
| 144 | 3600 | | | X | X | | | X | 448.3 | 11207 |

[A]An“ X” indicates standard corrugation sizes for each nominal diameter of pipe.
[B]Measured in valley of annular corrugations. Not applicable to helically corrugated pipe.
[C]Additional size for Type IS pipe.

### 5.3.3 Corrugated Aluminum

Aluminum was not identified as an elemental metal until 1807. Copper, bronze, iron, and steel have been in use for thousands of years. First refined in 1825, aluminum was considered a luxurious metal more expensive than gold. According to The Aluminum Association's article *History of Aluminum*, "Napoleon III, the first President of the French Republic, served his state dinners on aluminum plates. Rank-and-file guests were served on dishes made with gold or silver." It was not until the late 1800s that the development of commercial production of aluminum became affordable. (Aluminum Association n.d.)

In 1965, Purdue University published the report *Aluminum Pipe Culverts* at the request of the Indiana State Highway Department. The purpose of the report was to provide general practices and policies regarding the use of corrugated aluminum culvert pipe. "Although aluminum has been used extensively in the construction industry for several decades," the report states, "its advent into the culvert pipe market is relatively new." This fact remains true as little research is available for aluminum culverts compared to the vast amount of documented studies available on reinforced concrete culverts and corrugated steel culverts. (Brown 1965)

According to Figure 5-1, the top five most widely used culvert materials in North America include: concrete, galvanized steel, high density polyethylene, polyvinyl chloride, and aluminized steel. Aluminum is considered the sixth most widely used culvert material by transportation agencies along with polymer coated steel. The only culvert materials that were used less than aluminum are vitrified clay, ductile iron, and fiber glass. The results shown in Figure 5-1 were obtained from a survey conducted in 2014.

Corrugated aluminum culverts must meet the requirements of ASTM B745 (or AASHTO M196) *Standard Specification for Corrugated Aluminum Pipe for Sewers and Drains*. Corrugated aluminum pipe is classified similar to corrugated steel pipe. An example of corrugated aluminum pipe sizes is shown in Table 5-4. The classifications considered in this project include: Type I, Type IA, and Type IR. According to ASTM B745, Type I, Type IA, and Type IR are applicable to pipe having a full circular cross section. However, each type has different corrugations. Type I is fabricated with annular or helical corrugations. Type I has only a single thickness of corrugated sheet. Type IA pipe has an outer shell of corrugated sheet and an inner liner of uncorrugated sheet. Type IR is fabricated with helical ribs projecting outwardly. Type IR pipe has only a single thickness of uncorrugated sheet.

## Table 5-4: Corrugated Aluminum Pipe Sizes (ASTM B745)

| Nominal Inside Diameter | | Corrugation Sizes[A] | | | | | Minimum Outside Circumference[B] | |
|---|---|---|---|---|---|---|---|---|
| in. | mm | 1½ by ¼ in. 38 by 6.5 mm | 2⅔ (by ½ in. 68 by 13 mm | 3 by 1 in. 75 by 25 mm | 6 by 1 in. 150 by 25 mm | Ribbed Pipe[C] | in. | mm |
| 4 | 100 | X | | | | | 11.4 | 284 |
| 6 | 150 | X | | | | | 17.7 | 441 |
| 8 | 200 | X | | | | | 24.0 | 598 |
| 10 | 250 | X | | | | | 30.2 | 755 |
| 12 | 300 | | X | | | | 36.5 | 912 |
| 15 | 375 | | X | | | X | 46.0 | 1148 |
| 18 | 450 | | X | | | X | 55.4 | 1383 |
| 21 | 525 | | X | | | X | 64.8 | 1620 |
| 24 | 600 | | X | | | X | 74.2 | 1854 |
| 27 | 675 | | X | | | X | 83.6 | 2091 |
| 30 | 750 | | X | X | | X | 93.1 | 2325 |
| 33 | 825 | | X | X | | X | 102.5 | 2561 |
| 36 | 900 | | X | X | | X | 111.9 | 2797 |
| 42 | 1050 | | X | X | | X | 130.8 | 3269 |
| 48 | 1200 | | X | X | X | X | 149.6 | 3739 |
| 54 | 1350 | | X | X | X | X | 168.4 | 4209 |
| 60 | 1500 | | X | X | X | X | 187.0 | 4675 |
| 66 | 1650 | | X | X | X | X | 205.7 | 5142 |
| 72 | 1800 | | X | X | X | X | 224.3 | 5609 |
| 78 | 1950 | | | X | X | X | 243.0 | 6075 |
| 84 | 2100 | | | X | X | X | 261.7 | 6542 |
| 90 | 2250 | | | X | X | | 280.3 | 7008 |
| 96 | 2400 | | | X | X | | 299.0 | 7475 |
| 102 | 2550 | | | X | X | | 317.6 | 7941 |
| 108 | 2700 | | | X | X | | 336.3 | 8408 |
| 114 | 2850 | | | X | X | | 355.0 | 8874 |
| 120 | 3000 | | | X | | | 373.6 | 9341 |

[A] An "X" indicates standard corrugation sizes for each nominal diameter of pipe.
[B] Measured in valley of annular corrugations. Not applicable to helically corrugated pipe.
[C] Rib sizes ¾ by ¾ by 7½ in. [19 by 19 by 190 mm] and ¾ by 1 by 11½ in. [19 by 25 by 292 mm].

5.3.4 High Density Polyethylene

The discovery of polyethylene was purely accidental. In 1933, two British chemists, Eric Fawcett and Reginald Gibson, were working with ethylene at high pressures when they created a solid form of polyethylene. According to the British Broadcasting Company article *History of the World: The First Piece of Polyethylene*, polyethylene proved to be a timely breakthrough.

> *By the start of World War II, large plants were busy producing large quantities of this new substance which proved invaluable to the war effort. Polyethylene was used as an insulating material for radar cables during World War II, and the substance was a closely guarded secret. Its availability gave Britain an advantage in long-distance air warfare, most significantly in the Battle of the Atlanta against the German submarines which threatened to starve Britain of food*. (BBC 2010)

In 1953, German chemists Karl Ziegler and Erhard Holzkamp invented high density polyethylene. In 1954, the Phillips Petroleum Company introduced high density polyethylene under the brand name Marlex® polyethylene. Marlex® polyethylene was used by Wham-O, an American toy manufacturer, to develop a large ring of plastic tubing that would eventually be inducted into the National Toy Hall of Fame.

> *The first Hula-Hoops were made from a patented plastic called Marlex and sold for $1.98. Amazingly, twenty million hoops were sold in the very first 6 months of production which ignited the Hula-Hoop craze of the 1950's. And in the first two years, we sold over a staggering 100 million of them*! (Wham-O, n.d.)

According to the Plastics Technology article *No. 3 - HDPE*, "It was this fad that led to large-volume manufacturing of extruded HDPE pipe for high-performance applications such as natural-gas distribution, handling mine tailings, and sewer lines"

(Plastics Technology 2005). In time, Marlex® also became the preferred plastic for baby bottles and for safe, shatterproof food containers (ACS 1999).

In 1967, the first corrugated polyethylene pipe was commercially produced in the United States. The pipe was 4-inches in diameter. According to the article, "A Brief History of the Development and Growth of the Corrugated Polyethylene Pipe Industry in North America" written by James B. Goddard, "The intended market was agricultural drainage to increase crop yields, replacing clay tile, which dominated the market at that time, but was cumbersome and costly to install" (Goddard 2011).

In the early 1970s, polyethylene pipes were installed as highway underdrains by the Iowa Department of Transportation and the Georgia Department of Transportation. Georgia was the first department of transportation to include corrugated polyethylene in their standard specifications. "In September of 1981, the Ohio Department of Transportation installed the first known corrugated polyethylene cross-drain culvert under a state highway" (Goddard 2011). "Since then, more high density polyethylene pipes have been used for drainage applications than all other types of plastic pipe combined" (PPI 2008).

High density polyethylene is a thermoplastic material composed of carbon and hydrogen atoms. Polyethylene is formed when methane gas is converted into ethylene. Polyethylene materials are classified as high density polyethylene, medium density polyethylene, and low density polyethylene. Medium density polyethylene is typical for low-pressure gas pipelines. Low density polyethylene is mainly used for small-diameter water-distribution pipes. High density polyethylene has greater density and strength than medium density polyethylene and low density polyethylene due to the branching of its molecular chain. A comparison of the three common polyethylene materials is shown in Table 5-5.

**Table 5-5: Density of Polyethylene Materials (PPI 2008)**

| Polyethylene Material | Density |
|---|---|
| High Density Polyethylene | 0.941 g/cm$^3$ ≤ density ≤ 0.965 g/cm$^3$ |
| Medium Density Polyethylene | 0.926 g/cm$^3$ ≤ density ≤ 0.940 g/cm$^3$ |
| Low Density Polyethylene | 0.910 g/cm$^3$ ≤ density ≤ 0.925 g/cm$^3$ |

According to the article *History and Physical Chemistry of HDPE*, "The property characteristics of polyethylene depend upon the arrangement of the molecular chains."

> *The number, size and type of these side chains determine, in large part, the properties of density, stiffness, tensile strength, flexibility, hardness, brittleness, elongation, creep characteristics, and melt viscosity that are the results of the manufacturing effort and can occur during service performance of polyethylene pipe*. (PPI 2008)

According to *History and Physical Chemistry of HDPE*, "Density, molecular weight, and molecular weight distribution dominate the resin properties that influence the manufacture of the polyethylene pipe and the subsequent performance of the pipe." The effects of density, melt index, and molecular weight distribution are shown in Table 5-6.

High density polyethylene is a viscoelastic material, which exhibit a nonlinear stress-strain relationship and are dependent on time. Steel and concrete, on the other hand, have a linear stress-strain relationship and will return to their original shape after unloading in the elastic range of stresses. A perfect example of viscoelastic behavior can be seen by Silly Putty. According to the article *The Nature of Polyethylene Pipe Failure*,

> *If this material is pulled apart quickly, it breaks in a brittle manner. If, however, it is pulled slowly apart the material behaves in a ductile manner and can be stretched almost indefinitely. Decreasing the temperature of*

*Silly Putty decreases the stretching rate at which it becomes brittle. Plastic designers are well aware that, in the short term, many polymers can endure strain levels of 300% or more. However, for long-term performance, the window for design strain is massively smaller*. (O'Connor 2012)

**Table 5-6: Effects of Density, Melt Index, and Molecular Weight Distribution (PPI 2008)**

| EFFECTS OF CHANGES IN DENSITY, MELT INDEX AND MOLECULAR WEIGHT DISTRIBUTION | | | |
|---|---|---|---|
| **Property** | **As Density Increases, Property:** | **As Melt Index Increases, Property:** | **As Molecular Weight Distribution Broadens, Property:** |
| **Tensile Strength (At Yield)** | Increases | Decreases | |
| **Stiffness** | Increases | Decreases Slightly | Decreases Slightly |
| **Impact Strength** | Decreases | Decreases | Decreases |
| **Low Temperature Brittleness** | Increases | Increases | Decreases |
| **Abrasion Resistance** | Increases | Decreases | |
| **Hardness** | Increases | Decreases Slightly | |
| **Softening Point** | Increases | | Increases |
| **Stress Crack Resistance** | Decreases | Decreases | Increases |
| **Permeability** | Decreases | Increases Slightly | |
| **Chemical Resistance** | Increases | Decreases | |
| **Melt Strength** | | Decreases | Increases |
| **Gloss** | Increases | Increases | Decreases |
| **Haze** | Decreases | Decreases | |
| **Shrinkage** | Decreases | Decreases | Increases |

Polyethylene is prone to slow crack growth through the pipe wall. According to the report *Plastic Pipe Failure, Risk, and Threat Analysis*, "Slow crack growth failures occur over long periods of time at relatively low loads below the yield point of the material and are characterized by brittle fractures which exhibit very little material flow or deformation" (Gas Technology Institute 2009). ASTM F2136 *Standard Test Method for Notched, Constant Ligament-Stress (NCLS) Test to Determine Slow-Crack-Growth Resistance of HDPE Resins or HDPE Corrugated Pipe* may be used to determine the susceptibility of high density polyethylene pipe.  Figure 5-4 shows an optical micrograph of the slow crack growth failure process in polyethylene.

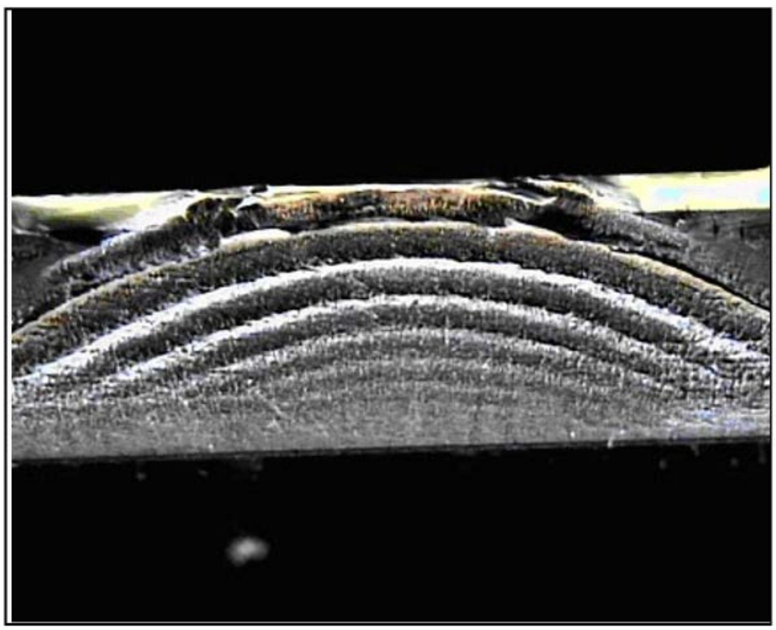


**Figure 5-4: Slow Crack Growth Failure Morphology (Gas Technology Institute 2009)**

As described in *The Nature of Polyethylene Pipe Failure*,

> *These cracks can initiate at microscopic stress-raising flaws, inherent in the basic pipe product or, more likely, from defects. These brittle mechanical failures are typically slit-type fractures that lie parallel to the pipe's extrusion direction. Circumferential hoop stress in the pipe wall is the driving force for crack opening*. (O'Connor 2012)

In 2006, The University of South Carolina published the report *Specifications for Culvert Pipe used in SCDOT Highway Applications* (Gassman 2006). The purpose of the report was to improve the field performance of reinforced concrete pipe, corrugated aluminum pipe, and high density polyethylene pipe used in roadway applications. In an effort to determine the field performance of high density polyethylene pipe, 45 high density polyethylene pipes were installed and monitored in sideline and driveway applications. Results concluded that high density polyethylene is a suitable pipe material. However, several of the pipes had cracks, localized bulges, and excess deformations. As noted in the report,

> *Installation problems such as poor preparation of bedding soils, inappropriate backfill material, and inadequate backfill cover contributed to the excessive deflection and observed internal cracking in pipes with noted damage. Appropriate installation procedures are essential to achieving high quality performance*. (Gassman 2006)

Field investigations proved that the performance of flexible, high density polyethylene pipe is significantly dependent on installation technique. Recommendations to improve the performance of high density polyethylene pipe in South Carolina included (Gassman 2006):

1. Training maintenance crews in the laying of plastic pipe;
2. Following ASTM and AASHTO installation procedures;
3. Inspecting the pipes after installation;
4. Developing guidelines for pipe product approval.

In 2009, the Texas Parks and Wildlife Department was forced to replace two miles of high density polyethylene pipe after two sections of 60-inch and 48-inch diameter pipe collapsed under 10-to-17 feet of earth fill. The repair cost $3.3 million and delayed the project by more than a year. Investigators found that 11,000 feet of pipe had questionable structural integrity. The high density polyethylene pipe was replaced with reinforced concrete pipe. Dr. Patricia D. Galloway, CEO of Pegasus Global Holdings Inc., released the following statement in the article *HDPE Pipe Failure at Texas Fish Hatchery offers Costly Lessons*:

> *Because corrugated HDPE pipe is a flexible material, not an independent structure like RCP, up to 90 percent of its successful installation is driven by the soil envelope surrounding it. It's imperative that the design firm and the installing engineers account for a wide range of pipe-soil variables when dealing with HDPE, ranging from material properties to installation conditions to external loads, any of which can lead to catastrophic failure*. (ACPA 2010)

High density polyethylene culverts must meet the requirements of AASHTO M252 *Standard Specification for Corrugated Polyethylene Drainage Pipe* or AASHTO M294 *Standard Specification for Corrugated Polyethylene Pipe, 300- to 1500-mm Diameter*. Section 7.4 of AASHTO M294 requires high density polyethylene pipe to have a minimum pipe stiffness as shown in Table 5-7 at five percent deflections. According to AASHTO M294, corrugated polyethylene pipe is classified as follows:

- Type C – This pipe shall have a full circular cross section, with a corrugated surface both inside and outside. Corrugations shall be annular.
- Type CP – This pipe shall be Type C with perforations.
- Type S – This pipe shall have a full circular cross section, with an outer corrugated pipe wall and a smooth inner linear. Corrugations shall be annular.
- Type SP – This pipe shall be Type S with perforations.
- Type D – This pipe shall consist of an essentially smooth inner wall/liner braced circumferentially or spirally with projections or ribs joined to an essentially smooth outer wall.

- Type DP – This pipe shall be Type D with perforations.

**Table 5-7: Polyethylene Pipe Stiffness per Diameter (AASHTO M294)**

| Diameter | | Pipe Stiffness | |
|---|---|---|---|
| **mm** | **inch** | **kPa** | **psi** |
| 300 | 12 | 345 | 50 |
| 375 | 15 | 290 | 42 |
| 450 | 18 | 275 | 40 |
| 525 | 21 | 260 | 38 |
| 600 | 24 | 235 | 34 |
| 675 | 27 | 205 | 30 |
| 750 | 30 | 195 | 28 |
| 900 | 36 | 150 | 22 |
| 1050 | 42 | 140 | 20 |
| 1200 | 48 | 125 | 18 |
| 1350 | 54 | 110 | 16 |
| 1500 | 60 | 95 | 14 |

AASHTO M294 requires testing for stress crack resistance in accordance with ASTM F2136 with one modification. According to Section 9.5.1, "The applied stress for the NCLS test shall be 4100 kPa (600 psi)". AASHTO M294 also requires testing for pipe stiffness in accordance with ASTM D2412 *Standard Test Method for Determination of External Loading Characteristics of Plastic Pipe by Parallel-Plate Loading*. The Parallel-Plate Loading Test determines the load-deflection characteristics of plastic pipe. As defined in the standard, pipe stiffness is "the value obtained by dividing the force per unit length of specimen by the resulting deflection in the same units at the prescribed percentage deflection".

The modified Spangler equation shown below can be used to approximate deflections under earth loads. "Pipe stiffness also relates to handling and installation characteristics of a pipe during the very early stages of soil consolidation around the pipe" (ASTM D2412).

$$x = \frac{D_e \; K \; W_c}{0.149 \; PS + 0.061 E'} \qquad \text{(Equation 5-1)}$$

Where,

| | | |
|---|---|---|
| $x$ | = | pipe deflection, inch |
| $K$ | = | bedding constant |
| $W_c$ | = | vertical load per unit of pipe length, lbf/inch |
| $PS$ | = | pipe stiffness, lbf/inch/inch |
| $D_e$ | = | deflection lag factor |
| $E'$ | = | modulus of soil reaction, psi |

In addition, the measured value of pipe stiffness can be related to the true rigidity (EI) of the pipe provided that the pipe remains elliptical. As stated in Appendix X2 of ASTM D2412,

> *The EI of a pipe is a function of the material's flexural modulus (E) and the wall thickness (t) of the pipe. However, the quantities pipes stiffness (PS) and stiffness factor (SF) are computed values determined from the test resistance at a particular deflection. These values are highly dependent on the degree of deflection, for as the pipe deflects the radius of curvature changes. The greater the deflection at which PS or SF are determined, the greater the magnitude of the deviation from the true EI value*. (ASTM D2412)

**Figure 5-5: Type C HDPE Pipe Interior and Exterior Corrugations (PPI 2008)**

**Figure 5-6: Type S HDPE Pipe Corrugated Exterior and Smooth Interior (PPI 2008)**

### 5.3.5 Polyvinyl Chloride

PVC was first discovered at the end of the nineteenth century. The first polyvinyl chloride pipe was manufactured in Germany in 1935. The synthetic material was chosen for residential water distribution and waste pipelines, because of its chemical resistance and smooth interior surface. The material also left no trace of taste or odor in the water supply. During the early years of manufacturing polyvinyl chloride pipe, more than 400 homes in central Germany were installed with PVC pipes. The use and availability of PVC pipe has steadily grown since the 1950s. With the ability to produce larger diameter pipe, PVC pipe has expanded into gravity storm sewers and highway drainage (Uni-Bell 2013; Stuart et al. 2011).

AASHTO M304 Poly(Vinyl Chloride) (PVC) Profile Wall Drain Pipe and Fittings Based on Controlled Inside Diameter requires that the pipes be made of PVC plastic that has a minimum cell classification of 12454C or 12364C as defined in ASTM D1784. The difference between these two classifications of PVC pipe is that one has a lower modulus and a higher strength, while the other has a higher modulus and a lower strength (McGrath et al. 2009). ASTM D1784 Standard Specification for Rigid Poly(Vinyl Chloride) (PVC) Compounds and Chlorinated Poly(Vinyl Chloride) (CPVC) Compounds gives requirements for impact resistance, tensile strength, modulus of elasticity, and deflection temperature under load. The following properties and their corresponding cell limits can be seen in Table 5-8. Similarly to HDPE, PVC’s classes are designated by the cell number for each property in the order that they are listed in Table 5-8. Figure 5-7 shows an example of how to read the above table using the Class 12454. (Stuart et al. 2011)

## Table 5-8: Class Requirements for PVC Compounds (ASTM D1784)

| Designation Order No. | Property and Unit | Cell Limits | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 0 | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 |
| 1 | Base resin | unspecified | poly(vinyl chloride) homo-polymer | chlorinated poly (vinl chloride) | vinl co-polymer | | | | | | | | |
| 2 | Impact resistance (izod), min: | | | | | | | | | | | | |
| | J/m of notch under notch | unspecified | <34.7 | 34.7 | 80.1 | 266.9 | 533.8 | 800.7 | | | | | |
| | ftlb/in. of notch under notch | | <0.65 | 0.65 | 1.5 | 5.0 | 10.0 | 15.0 | | | | | |
| 3 | Tensile strength min: | | | | | | | | | | | | |
| | Mpa | unspecified | <34.5 | 34.5 | 41.4 | 48.3 | 55.2 | | | | | | |
| | psi | | <5 000 | 5 000 | 6 000 | 7 000 | 8 000 | | | | | | |
| 4 | Modulus of elasticity in tension, min: | | | | | | | | | | | | |
| | Mpa | unspecified | <1930 | 1930 | 2206 | 2482 | 2758 | 3034 | | | | | |
| | psi | | <280 000 | 280 000 | 320 000 | 360 000 | 400 000 | 440 000 | | | | | |
| 5 | Deflection temperature under load, min, 1.82 Mpa [264 psi]; | | | | | | | | | | | | |
| | °C | unspecified | <55 | 55 | 60 | 70 | 80 | 90 | 100 | 110 | 120 | 130 | 140 |
| | °F | | <131 | 131 | 140 | 158 | 176 | 194 | 212 | 230 | 251 | 266 | 284 |
| | Flammability | A | A | A | A | A | A | A | A | A | A | A | A |

A - All compounds covered by this specification, when tested in accordance with Test Method D 635, shall yield the following results: average extent of burning of <25 min, average time of burning of <10 s.

Class Identification: 1 2 4 5 4

Poly(vinyl chloride) homopolymer — 1

Property and Minimum Value

Izod (34.7 J/m (0.65 ftlbf/in.)) under notch — 2

Tensile strength (48.3 Mpa (7000psi)) — 4

Modulus of elasticity in tension (2758 Mpa (400 000psi)) — 5

Deflection temperature under load (70°C (158°)) — 4

Figure 5-7: Cell Classification Key (ASTM D1784)

According to the article “Photostabilization of Poly(Vinyl Chloride) – Still on the Run” by Ali Hasan and Emad Yousif, polyvinyl chloride is the second largest manufactured resin by volume worldwide. “The polymer can be converted into many different products exhibiting an extremely wide range of properties both physical and chemical by using modifying agents, such as plasticizers, fillers and stabilizers” (Hasan and Yousif 2015). Polyvinyl chloride is mainly used for pipes and sidings in North America. In Europe and Asia, polyvinyl chloride is most commonly used for pipes and window frames. Intriguingly, the demand for polyvinyl chloride is much higher in developing countries like China and India while the material has seen a decline in industrialized countries. Figure 5-8 illustrates the world consumption of polyvinyl chloride in 2007. Table 5-9 lists some representative products manufactured from polyvinyl chloride.

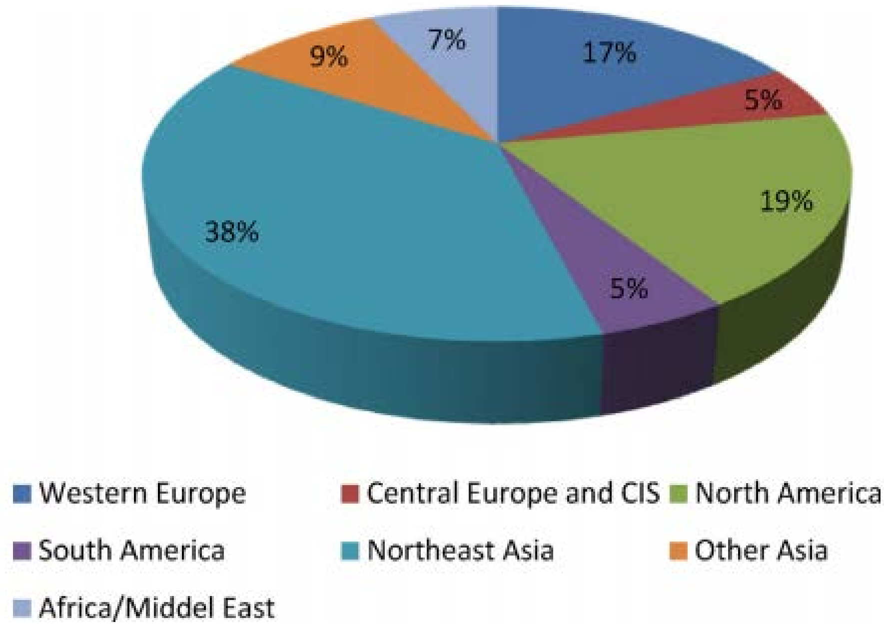



**Figure 5-8: World Consumption of Polyvinyl Chloride in 2007 (Hasan and Yousif 2015)**

**Table 5-9: Examples of Polyvinyl Chloride Applications (Whitfield and Associates 2008)**

| Construction | Automotive |
|---|---|
| Piping and fittings for water distribution, irrigation and sewers; grey water recycling kits; electrical conduits; siding, awnings, soffit, skirting, weather stripping, gutters and downspouts; decking and fencing; window, door frames and cladding; landfill liners and geomembranes; swimming pool liners; single-ply roofing; conveyor belts; piping used in food processing, chemical processing and other manufacturing; floor and wall coverings; coated paneling; adhesives; maintenance coatings | Interior upholstery; "soft" dashboard and arm rests, dashboard instrument components, airbag covers; body side moldings, bumper guards; windshield system components, rearview mirror housings; under-the-hood wiring; under-the-car abrasion coatings; floor mats; adhesives and sealants; boots and bellows; battery separators; audio and video components; lighting components; steering cover and transmission parts; A/C system components |
| **Medical and Healthcare** | **Electrical and Electronics** |
| Blood bags and tubing; cannulae; caps; catheters; connectors; cushioning products; device packages; dialysis | Computer housing and cabling; printed circuit board trays; power wire insulation and sheathing; communication cable |

| | |
|---|---|
| equipment and tubing; drainage tubing; drip chambers; ear protection; goggles; inflatable splints; inhalation masks; IV containers and components; laboratory ware; masks; mouthpieces; oxygen delivery components; seals; surgical wire; jacketing; thermal blankets; urine and colostomy bags; valves and fittings | jacketing; backing for power cable; electrical plugs and connectors, wall plates, connection boxes; soft keyboards; keyboard trays; coating for optical mouse pads; memory stick and USB covers/casings; LED product components; laminate for plastic security passes and "smart cards" |
| **Packaging** | **Consumer Products and Other** |
| Sterile medical packaging; tamper-proofing over-the-counter medication; shrink wrap for software, games, and household products; blister and clamshell packaging to protect toys, hardware, electronics, personal care products, and foods such as eggs and meat; bottles for household and personal care products, cooking oils and automotive lubricants; closures for bottles and jars; can coatings | Wind turbine blades; machinery parts; housings and handles for tools; garden hoses; tarpaulins; patio furniture, upholstery; appliance housings; window shades and blinds; table cloths, place mats, shower curtains; sporting goods, beach balls; vinyl leather goods; luggage, footwear, gloves, rainwear; handbags; apparel; coated paper; holiday decorations; toys |

In 2009, Bob Drake of *Civil + Structural Engineer News* presented the report *"Pipe Choices"* to aid in the selection of pipe materials, applications, and suppliers. "National associations representing the eight most commonly used types of pipe – concrete, corrugated steel, ductile iron, fiberglass, polyethylene, polyvinyl chloride, steel, and vitrified clay – provided responses to four questions" (Drake 2009). Uni-Bell PVC Pipe Association (2013) contributed information regarding polyvinyl chloride. The questions and their corresponding answers are shown below.

**Question 1: What applications and environments particularly favor the use of PVC pipe?**

> *Since PVC is a non-conductor, both galvanic and electrochemical effects are non-existent. PVC suffers no damage from attack of normal or corrosive soils. PVC offers much higher tensile strength than other thermoplastic pipe materials. PVC has superior stiffness compared with*

*other thermoplastic pipes, which enables PVC pipe to better resist buckling and limit long-term deformation under soil load.*

**Question 2: How have the performance and/or application of PVC pipe changed during the last five to 10 years?**

*Professor Al Moser of Utah State University performed strain tests on PVC pipe. After 22 years under test, Moser reported that no failures occurred. Tests demonstrated that under a constant strain condition, if the initial strain can be achieved, failure will not occur in PVC pipe*.

**Question 3: How does PVC pipe contribute to a project's green building or Leadership in Energy and Environmental Design (LEED) certification goals?**

*PVC pipes are not reactive with water and consequently do not adversely change water quality. While there are some chemical solvents that can alter or attack PVC pipe, the concentrations required to do so are much higher than those normally encountered in pipeline installations*.

**Question 4: What specific resources are available to help civil engineers?**

*The Uni-Bell PVC Pipe Association's website provides access to available technical information, software, and its comprehensive Handbook of PVC Pipe – Design and Construction*.

Solid wall polyvinyl chloride must meet the requirements of AASHTO M278 *Class PS46 Poly(Vinyl Chloride) (PVC) Pipe* and ASTM F679 *Standard Specification for Poly(Vinyl Chloride) (PVC) for Large-Diameter Plastic Gravity Sewer Pipe and Fittings*. Profile wall polyvinyl chloride pipe must meet the requirements of AASHTO M304 *Poly(Vinyl Chloride) (PVC) Profile Wall Drain Pipe and Fittings Based on Controlled Inside Diameter*. The *Handbook of PVC Pipe Design and Construction* explains the differences between solid wall and profile pipe as follows:

*PVC solid-wall pipe takes the form of a cylinder with homogeneous walls of uniform thickness. Both the interior and exterior surfaces are smooth. PVC profile-wall pipe is pipe that has a smooth internal surface but a non-solid wall cross-sectional pattern perpendicular to the pipe axis. Wall patterns may be open-profile (such as concentric ribs or spiral ribs) or* *closed-profile (smoother inner and outer walls with internal ribs). Profile-wall pipes are not intended for pressure service*. (Uni-Bell 2013)

According to Section 6.4 of AASHTO M278, the minimum pipe stiffness values for solid wall polyvinyl chloride shall be 320 kilopascals when tested in accordance with ASTM D2412. However, the minimum pipe stiffness values vary for profile wall pipe. Table 5-10 illustrates the minimum pipe stiffness as well as the required pipe dimensions for profile wall polyvinyl chloride pipe.

**Table 5-10: Profile Wall Pipe Requirements (AASHTO M304)**

| Nominal Pipe Size, mm | Min. Average Inside Diameter, mm | Min. Waterway Wall, mm | Min. Pipe Impact Strength, J | Min. Pipe Stiffness, kPa |
|---|---|---|---|---|
| 100 | 100.0 | 0.56 | 108 | 318 |
| 150 | 149.7 | 0.64 | 108 | 318 |
| 200 | 199.7 | 0.89 | 136 | 318 |
| 250 | 249.6 | 1.14 | 136 | 318 |
| 300 | 296.8 | 1.52 | 136 | 318 |
| 375 | 363.3 | 1.90 | 136 | 262 |
| 450 | 444.8 | 2.16 | 136 | 221 |
| 525 | 624.7 | 2.54 | 136 | 193 |
| 600 | 594.7 | 2.92 | 136 | 165 |
| 675 | 669.8 | 3.18 | 136 | 152 |
| 750 | 746.4 | 3.43 | 163 | 131 |
| 900 | 898.4 | 3.94 | 163 | 110 |
| 1050 | 1050.9 | 4.32 | 163 | 97 |
| 1200 | 1202.9 | 4.83 | 163 | 83 |

According to WSDOT'S *Hydraulics Manual* (2008) solid wall polyvinyl chloride culvert pipe shall have a smooth interior and conform to the requirements of ASTM D3032 for pipes up to 15 inches in diameter and ASTM F679 for pipe sizes 18 to 27 inch.

The culvert pipes conforming to ASTM F679 shall be Type 1 only. The Hydraulics Manual permits use of both types of pipe beneath all state highways.

WSDOT's *Hydraulics Manual* also permits use of profile wall polyvinyl chloride culvert pipe having a smooth waterway wall braced circumferentially or spirally with projections or ribs. "The pipe may have an open profile, where the ribs are exposed, or the pipe may have a closed profile, where the ribs are enclosed in an outer wall" (WSDOT 2008). Profile wall culvert pipe shall meet the requirements of AASHTO M304 or ASTM F794, Series 46. The Hydraulics Manuals permits use of these type of pipes beneath all state highways. Figure 5-9 shows the typical polyvinyl chloride pipe profile wall cross sections.

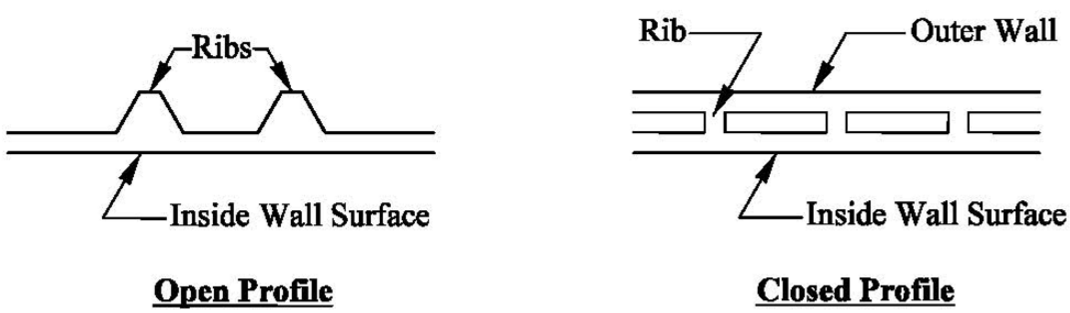
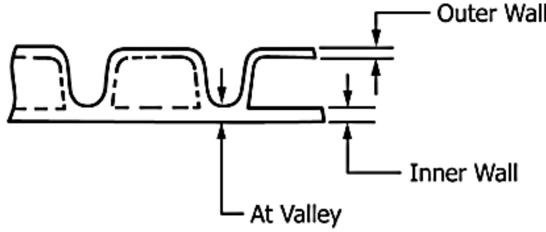



**Figure 5-9: Typical Profile Wall Polyvinyl Chloride Pipe Cross Sections (WSDOT 2008)**

However, there are several state departments of transportation that specify a different standard specification for polyvinyl chloride culvert pipe. According to NCDOT's *Drainage User Manual*, "… corrugated PVC pipe material shall meet the product specifications of ASTM F949 and shall have a smooth interior" (NCDOT 2003). Likewise, FDOT bases the service life of polyvinyl chloride pipe according to which ASTM standard specification the pipe material meets.

ASTM F949 *Standard Specification for Poly(Vinyl Chloride) (PVC) Corrugated Sewer Pipe with a Smooth Interior and Fittings* provides requirements for profile wall pipe having an outer corrugated wall fused to a smooth inner wall. Pipe stiffness shall be a minimum of 46 psi or 115 psi. Table 5-11 depicts the required pipe dimensions based on a pipe stiffness of 46 psi and 115 psi.

## Table 5-11: Profile Wall Pipe Dimensions Based on Pipe Stiffness (ASTM F949)




For Pipe Stiffness of 46 PSI

| Nominal Size in. | Outside Diameter | | Inside Diameter | | Minimum Wall Thickness | | |
|---|---|---|---|---|---|---|---|
| | Average, in. (mm) | Tolerance on Average, in. (mm) | Average, in. (mm) | Tolerance on Average, in. (mm) | Inner Wall, in. (mm) | Outer Wall, in. (mm) | At Valley, in. (mm) |
| 4 | 4.300 (109.2) | ±0.009 (±0.229) | 3.950 (100.3) | ±0.011 (±0.279) | 0.022 (0.559) | 0.018 (0.457) | 0.028 (0.711) |
| 6 | 6.420 (163.1) | ±0.011 (±0.279) | 5.909 (150.1) | ±0.015 (±0.381) | 0.025 (0.635) | 0.022 (0.559) | 0.032 (0.813) |
| 8 | 8.600 (218.4) | ±0.012 (±0.305) | 7.881 (200.2) | ±0.018 (±0.457) | 0.035 (0.889) | 0.030 (0.762) | 0.045 (1.143) |
| 10 | 10.786 (273.9) | ±0.015 (±0.381) | 9.846 (250.1) | ±0.021 (±0.533) | 0.045 (1.143) | 0.036 (0.914) | 0.055 (1.397) |
| 12 | 12.795 (325.0) | ±0.018 (±0.457) | 11.715 (297.6) | ±0.028 (±0.711) | 0.058 (1.397) | 0.049 (1.245) | 0.072 (1.829) |
| 15 | 15.658 (397.7) | ±0.023 (±0.584) | 14.338 (364.2) | ±0.035 (±0.889) | 0.077 (1.956) | 0.055 (1.397) | 0.092 (2.337) |
| 18 | 19.152 (486.5) | ±0.028 (±0.711) | 17.552 (445.8) | ±0.042 (±1.067) | 0.084 (2.134) | 0.067 (1.702) | 0.103 (2.616) |
| 21 | 22.630 (574.8) | ±0.033 (±0.838) | 20.705 (525.9) | ±0.049 (±1.24) | 0.095 (2.413) | 0.073 (1.854) | 0.110 (2.800) |
| 24 | 25.580 (649.7) | ±0.039 (±0.991) | 23.469 (596.1) | ±0.057 (±1.448) | 0.110 (2.791) | 0.085 (2.161) | 0.123 (3.124) |
| 27 | 28.860 (733.0) | ±0.049 (±1.25) | 26.440 (671.6) | ±0.069 (±1.75) | 0.120 (3.048) | 0.091 (2.311) | 0.137 (3.486) |
| 30 | 32.150 (816.6) | ±0.059 (±1.50) | 29.469 (748.5) | ±0.081 (±2.057) | 0.130 (3.302) | 0.105 (2.667) | 0.147 (3.734) |
| 36 | 38.740 (984.0) | ±0.079 (±2.007) | 35.475 (901.1) | ±0.105 (±2.667) | 0.150 (3.810) | 0.125 (3.175) | 0.171 (4.343) |
| 42 | 45.800 (1163.3) | ±0.093 (±2.36) | 41.500 (1054.1) | ±0.127 (±3.23) | 0.160 (4.06) | 0.135 (3.43) | 0.188 (4.78) |
| 48 | 52.800 (1341.1) | ±0.108 (±2.74) | 47.500 (1206.5) | ±0.143 (±3.63) | 0.165 (4.19) | 0.140 (3.56) | 0.195 (4.95) |

For Pipe Stiffness of 115 PSI

| Nominal Size | Outside Diameter | | Inside Diameter | | Minimum Wall Thickness | | |
|---|---|---|---|---|---|---|---|
| | Average, in. (mm) | Tolerance on Average, in. (mm) | Average, in. (mm) | Tolerance on Average, in. (mm) | Inner Wall, in. (mm) | Outer Wall, in. (mm) | At Valley, in. (mm) |
| 8 | 8.600 (218.4) | ±0.012 (±0.305) | 7.710 (195.8) | ±0.018 (±0.457) | 0.037 (0.940) | 0.050 (1.270) | 0.048 (1.219) |
| 10 | 10.786 (273.9) | ±0.015 (±0.381) | 9.644 (245.0) | ±0.021 (±0.533) | 0.046 (1.295) | 0.052 (1.320) | 0.065 (1.651) |
| 12 | 12.795 (325.0) | ±0.018 (±0.457) | 11.480 (291.6) | ±0.028 (±0.711) | 0.070 (1.778) | 0.068 (1.727) | 0.091 (2.311) |
| 15 | 15.658 (397.7) | ±0.023 (±0.584) | 14.053 (356.97) | ±0.035 (±0.889) | 0.092 (2.337) | 0.088 (2.235) | 0.118 (2.997) |

According to ASTM F949, profile wall pipes have thicker walls or deeper profiles than solid wall pipes regardless of pipe stiffness or material. Therefore, greater bending strains develop in profile wall pipes than solid wall pipes when flattened to the same degree. The two equations shown below may be used to calculate the bending strains for profile wall pipes and solid wall pipes.

$$\varepsilon_p = D_f \;\; \Delta p \left( \frac{C_{\max}}{R} \right) \qquad \text{(Equation 5-2)}$$

$$\varepsilon_s = D_f \;\; \Delta s \left( \frac{0.5t}{R} \right) \qquad \text{(Equation 5-3)}$$

Where,

| | | |
|---|---|---|
| $\varepsilon_p$ | = | bending strain in the profile wall pipe |
| $\varepsilon_s$ | = | bending strain in an equal stiffness solid wall pipe |
| $\Delta p$ | = | fattening level in the profile wall pipe |
| $\Delta s$ | = | fattening level in a solid wall pipe |
| $D_f$ | = | factor to account for the shape of the deflected pipe |
| $C_{max}$ | = | extreme fiber distance of the pipe wall |
| $t$ | = | thickness of solid wall pipe wall |
| $DR$ | = | dimension ratio of a solid wall pipe |
| $R$ | = | mean pipe radius |

### 5.3.6 Polypropylene

The discovery of polypropylene occurred almost simultaneously in the United States and in Europe. As World War II ended, the Phillips Petroleum Company looked for ways to expand its product line as the wartime demand for oil diminished. J. Paul Hogan and Robert L. Banks, two researchers working for the Phillips Petroleum Company, were asked to find ways to convert propylene and ethylene into gasoline. Instead, they discovered crystalline polypropylene. The American Chemical Society credits J. Paul Hogan and Robert L. Banks with the discovery of polypropylene. Both researchers were posthumously inducted into the Plastics Hall of Fame.

Despite being used for underground drainage and sewage applications in Europe for approximately 30 years, polypropylene pipe is relatively new to North America. According to Borealis, a European polyolefin product manufactuer, “It [polypropylene] began being specified for the production of sewerage pipes from the 1970’s. Since 1950, there has been an average global increase of 9% per year in the production and consumption of plastics” (Borealis 2010). The demand for plastic gravity pipe systems in Europe is shown in Figure 5-10.

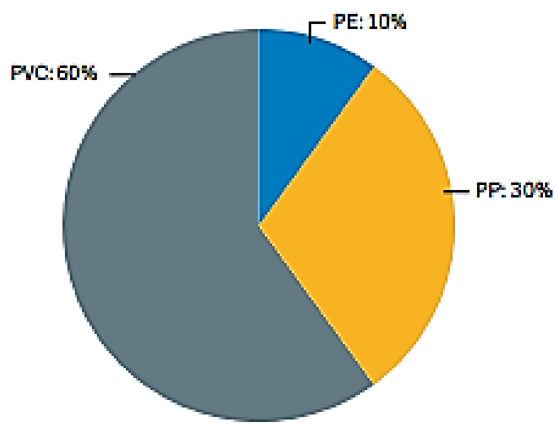



**Figure 5-10: Demand for Plastic Gravity Pipe Systems in Europe (Borealis 2010)**

As with polyethylene and polyvinyl chloride, polypropylene is also a viscoelastic material and has properties between that of high density polyethylene and low density polyethylene. According to the report *Evaluation of Polypropylene Drainage Pipe*, “In general, polypropylene exhibits excellent mechanical and chemical characteristics, including high strength, high stiffness, high resistance to stress crack propagation, and high chemical resistance. Because of its relatively high strength to weight ratio, it is more rigid than other polyolefin" (Hoppe 2011). Table 5-12 illustrates some of the mechanical properties for polyethylene pipe and polypropylene pipe.

**Table 5-12: Thermoplastic Pipe Mechanical Properties (ADS 2015)**

| ADS Product | Material | Allowable Strain, % | Initial | | 75-Year | |
|---|---|---|---|---|---|---|
| | | | $F_u$ psi | E psi | $F_u$ psi | E psi |
| N-12 ST IB, WT IB, Plain End, SaniTite, Low Head | Polyethylene | 5 | 3,000 | 110,000 | 900 | 21,000 |
| NP-12 HP Storm and SaniTite HP Sanitary | Polypropylene | 4 | 3,500 | 175,000 | 1,000 | 28,000 |

Section 12 of the *AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing* (AASHTO 2008) requires the material properties shown in Table 5-13 for the design of thermoplastic pipe. According to Section 12.12.3.3, it is the responsibility of the engineer to choose between the initial and the 50-year mechanical property requirements. However, buckling must be based on the 50-year value for modulus of elasticity. As stated in the commentary,

> *The PE and PVC materials described herein have stress/strain relationships that are nonlinear and time-dependent. The 50-year design tensile strength requirements are derived from hydrostatic design models and indicate a minimum 50-year life expectancy under continuous application of that stress. The 50-year moduli of elasticity do not indicate a softening of the pipe material but the time-dependent relation between stress and strain*. (AASHTO 2008)

**Table 5-13: Thermoplastic Pipe Mechanical Properties (AASHTO 2008)**

| Type of Pipe | Minimum Cell Class | Service Long-Term Tension Strain Limit, $\varepsilon_{yt}$ (%) | Factored Compr. Strain Limit, $\varepsilon_{yc}$ (%) | Initial | | 50-yr | | 75-yr | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | $F_u$ min (ksi) | $E$ min (ksi) | $F_u$ min (ksi) | $E$ min (ksi) | $F_u$ min (ksi) | $E$ min (ksi) |
| Solid Wall PE Pipe – ASTM F714 | ASTM D3350, 335434C | 5.0 | 4.1 | 3.0 | 110.0 | 1.44 | 22 | 1.40 | 21 |
| Corrugated PE Pipe – AASHTO M 294 | ASTM D3350, 435400C | 5.0 | 4.1 | 3.0 | 110.0 | 0.90 | 22 | 0.90 | 21 |
| Profile PE Pipe – ASTM F894 | ASTM D3350, 334433C | 5.0 | 4.1 | 3.0 | 80.0 | 1.12 | 20 | 1.10 | 19 |
| | ASTM D3350, 335434C | 5.0 | 4.1 | 3.0 | 110.0 | 1.44 | 22 | 1.40 | 21 |
| Solid Wall PVC Pipe – AASHTO M 278, ASTM F679 | ASTM D1784, 12454C | 5.0 | 2.6 | 7.0 | 400.0 | 3.70 | 140 | 3.60 | 137 |
| | ASTM D1784, 12364C | 3.5 | 2.6 | 6.0 | 440.0 | 2.60 | 158 | 2.50 | 156 |
| Profile PVC Pipe – AASHTO M 304 | ASTM D1784, 12454C | 5.0 | 2.6 | 7.0 | 400.0 | 3.70 | 140 | 3.60 | 137 |
| | ASTM D1784, 12364C | 3.5 | 2.6 | 6.0 | 440.0 | 2.60 | 158 | 2.50 | 156 |

According to *Evaluation of Polypropylene Drainage Pipe*, "Polypropylenes exhibit higher tensile, flexural, and compressive strength and higher moduli than polyethylene" (Hoppe 2011). Table 5-14 provides typical uses for polypropylene and high density polyethylene.

**Table 5-14: Uses for Polypropylene and High Density Polyethylene (ACS 1999)**

| Industry | Polypropylene | High Density Polyethylene |
|---|---|---|
| Automotive | Battery Cases and Trays<br>Bumpers<br>Fender Liners<br>Interior Trim<br>Reservoirs | Fuel Tanks<br>Motor Oil Containers<br>Portable Gas Cans<br>Under-hood Reservoirs<br>Wire Insulation |
| Education | Binders<br>Transparent Sleeves<br>Writing Instruments | Classroom/ Stadium Seating<br>Notebook Binders |
| Environment | Geotextiles for Erosion<br>Pavement Under-liners | Chemical Toilets<br>Erosion Barriers<br>Landfill Liners<br>Pond and Canal Liners |
| Home | Appliance Housings<br>Bottles and Containers<br>Food Packaging<br>Microwave Cookware | Food and Drink Containers<br>Household Product Bottles<br>Outdoor Furniture<br>Toys<br>Trash and Lawn Bags |
| Industry | Carpeting<br>Crates and Trays<br>Filters<br>Office Furniture<br>Tapes<br>Woven Bags | Cable Jacketing<br>Oil and Gas Lines<br>Packaging Films<br>Tank and Drums<br>Wire Insulation |
| Medical | Medical Implements<br>Packaging<br>Syringes | Biomedical Waste Containers<br>Pharmaceutical Bottles<br>Tubing and Catheters |
| Municipal | Ropes and Twine | Highway Barriers<br>Slip-lining for Sewers<br>Trash Containers<br>Utility Pipes |
| Recreation | Safety Equipment<br>Sporting Goods<br>Sportswear | Basketball Backboards<br>Water Bottles and Coolers<br>Watercraft Components |

The Virginia Center for Transportation Innovation and Research published the report *Evaluation of Polypropylene Drainage Pipe* for the Virginia Department of

Transportation (VDOT). The purpose of the work was to conduct a field evaluation to assess the potential suitability of polypropylene pipe for cross-drain applications. The VDOT selected five test locations on low-volume rural roads. Average daily traffic counts were obtained from the 2007 VDOT traffic survey and indicated that there were: 910 vehicles per day on Route 684, 60 vehicles per day on Route 736, 570 vehicles per day on Route 635, and 90 vehicles per day on Route 698. A summary of the pipe installations is shown in Table 5-15. Polypropylene pipes replaced the existing corrugated metal pipes and concrete pipes that had reached the end of their service life. American Drainage Systems Inc. (ADS) supplied the dual- and triple-wall plastic pipes with nominal diameters of 30 and 48 inches (Figure 5-11). The pipes manufactured by ADS were part of ADS' N-12 High Performance product line that was specifically designed for gravity flow and sanitary sewer applications. The stiffness of the pipe was 46 psi at 5 percent deflection, Manning's n value was 0.012, and the cover height was approximately 2 feet. The results concluded that after one year of service, the maximum deformations of all pipes were less than 5 percent. This satisfied current VDOT post-installation inspection requirements. The report noted that no signs of crushing, buckling, or material degradation were detected. (Hoppe 2011)

Table 5-15: Summary of Pipe Installation at Test Sites (Hoppe 2011)

| Route | Diameter | Pipe Wall | Length | Vehicle per Day |
|---|---|---|---|---|
| 684 | 48-inch | Triple | 55.0 feet | 910 |
| 736 | 30-inch | Dual | 50.0 feet | 60 |
| 635 | 30-inch | Dual | 30.0 feet | 570 |
| 635 | 48-inch | Triple | 33.5 feet | |
| 698 | 48-inch | Triple | 31.2 feet | 90 |

**Figure 5-11: Dual-Wall and Triple-Wall Profiles of ADS Pipes (Hoppe 2011)**

Weather conditions were severe during the field evaluations. This resulted in substantial precipitation and unusually low ambient air temperatures. Based on visual observations and cross-sectional measurements, the report concluded that all polypropylene pipes performed satisfactory in the first year of service and were found to be fully functional. Other observations made by the report include:

1. Polypropylene is lightweight and easy to handle, assemble and install.
2. It does not require any specialized equipment or methods during construction.
3. When tested at Route 698, the pipe showed no evidence of wear and erosion from a relatively high water flow combined with the substantial presence of large rock particles.
4. Its double seal design reduced the risk of a joint leakage.

### 5.4 Flexible and Rigid Culverts

Culverts are separated into two distinct categories: flexible and rigid. According to the ACPA, flexible pipe is at least 95% dependent on soil support and the installation expertise of the contractor. As stated in the Plastics Pipe Institute's (PPI) *Design Methodology*, "When flexible pipe deflects against the backfill, the load is transferred to and carried by the backfill. When loads are applied to rigid pipe, on the other hand, the load is transferred through the pipe wall into the bedding" (PPI 2008). Backfill quality and compaction are the most important factors in ensuring satisfactory performance of flexible pipe (Zhao et al. 1998).

As stated in *Cross-Drain Pipe Material Selection Algorithm* (French 2013), "A flexible pipe's ability to deflect under loads without any structural damage when installed properly is often beneficial in deep installations". Figure 5-12 illustrates a flexible pipes' and rigid pipes' response to loading. A flexible pipe can deflect at least two percent without cracking, rupture, or any other sign of structural distress. However, "due to loss of lateral support, partial excavation and exposure of flexible pipe is likely to result in excessive deformation, and may lead to collapse" (Zhao et al. 1998). Examples of flexible culverts include corrugated steel, corrugated aluminum, high density polyethylene, polyvinyl chloride and polypropylene.

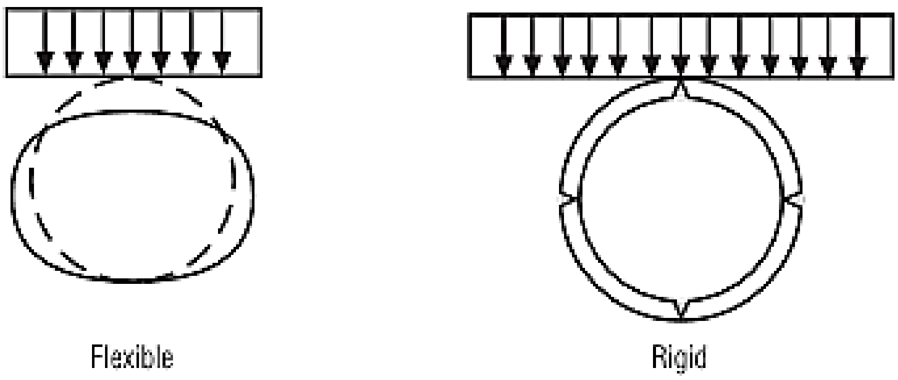



**Figure 5-12: Flexible and Rigid Pipe Response to Loading (PPI 2008)**

Rigid culverts include non-reinforced concrete, reinforced concrete, and clay. Concrete pipe is a rigid pipe system that is over 85% dependent on the pipe strength and only 15% dependent on the strength derived from the soil envelope. Rigid pipe is sometimes classified as pipe that cannot deflect more than 2% without significant structural distress such as cracking (PPI 2008). “Existing buried, rigid pipe is less sensitive to re-excavation and backfilling, because of its inherent strength” (Zhao et al. 1998). Figure 5-13 illustrates the difference in backfill interaction between a flexible pipe and a rigid pipe. The installation parameters of various pipes are shown in Table 5-16.

The Connecticut Department of Transportation (ConnDOT) *Drainage Manual* differentiates between flexible and rigid culvert behavior:

> *Flexible pipe has relatively little bending stiffness or bending strength of its own. As loads are applied to the culvert, the culvert attempts to deflect. In the case of a round pipe, the vertical diameter decreases and the horizontal diameter increases.*
>
> *The load carrying capacity of rigid culverts is essentially provided by the structural strength of the pipe and little benefit from the surrounding earth*

*is required. When vertical loads are applied to a rigid pipe, zones of tension and compression are created.* (ConnDOT 2000)

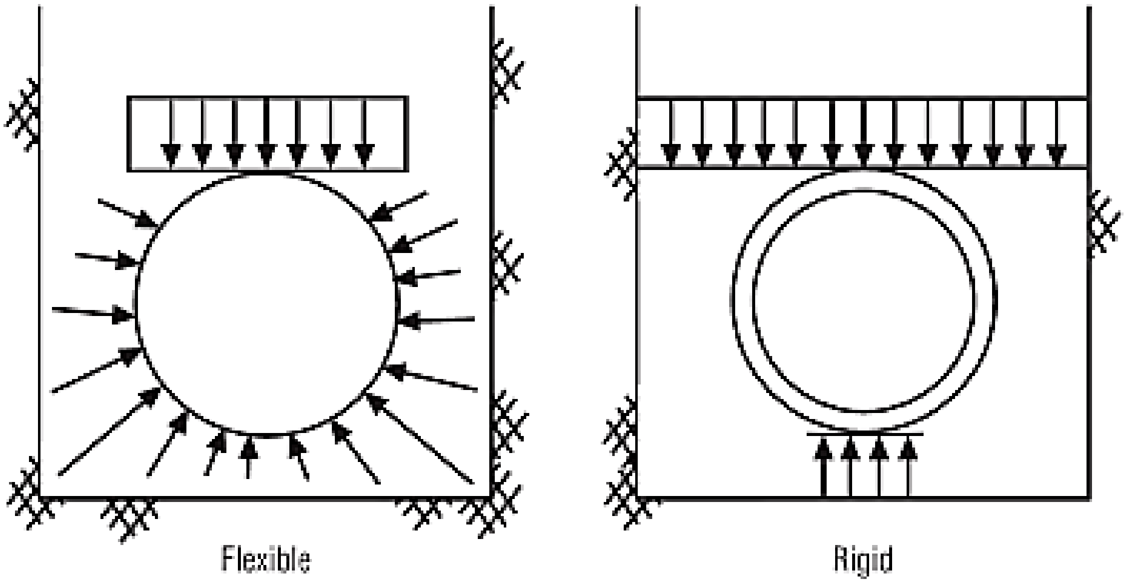



**Figure 5-13: Flexible and Rigid Pipe Backfill Interaction (PPI 2008)**

**Table 5-16: Installation Parameters of Various Pipes (Zhao et al. 1998)**

| Installation Parameter | Rigid Pipe | Flexible Pipe |
|---|---|---|
| Trench Width | As narrow as possible. Earth load increases as the trench width increases, until transition width. Less width required for work space. | Earth load does not increase with width beyond the prism limits. Sufficient width is required to carry out careful compaction. |
| Joints | Bell-spigot joints with gaskets. More joints due to short sections. | Plastic Pipe: Elastomeric seal or solvent cement. Easy cutting for length adjustment/ fewer joints.<br>Corrugated Steel Pipe: Steel coupling bands with neoprene gaskets or bitumen sealants. Welding. |

| | | |
|---|---|---|
| Minimum Cover | 900 mm required before use of a heavy compactor. Damage due to compaction not reported. | Plastic Pipe: 900 mm required before use of a heavy compactor. Over compaction may cause excessive deflection.<br>CSP: Minimum cover ranging from 700 to 1400 mm. |
| Operation | May require additional equipment and manpower to handle heavier pipe sections. Requires less compaction effort. | Requires adequate on-site inspection. Requires maximum effort for effective compaction. Ease of transportation and handling. |
| Temperature Effects | Strength increases as temperature decreases in the range of -20 to -30°C. Impact strength also increases with decrease in temperature. | HDPE: Minimum installation temperature is -34°C. Impact strength is not affected significantly by low temperature.<br>PVC: Minimum installation temperature is -18°C. Impact strength is reduced by up to 30% when temperature decreases from 23 to 0°C. |
| UV Degradation | UV degradation is negligible. | Plastic Pipe: Susceptible to UV degradation in long-term exposure. |
| Adjacent Excavation | Less sensitive to re-excavation and backfilling. Depending on the location, partial exposure usually does not cause significant distress to pipes. | Once exposed, flexible pipe must be backfilled and compacted with great care, according to the original specifications to restore its strength. Partial excavation and exposure is likely to result in excessive deformation. |

Dr. Anson Marston at Iowa State University conducted a 21-year study to analyze soil pressures on buried culverts. He claimed that the “load on a rigid pipe would always be higher than the load on a flexible pipe due to the differences in interaction between each type of pipe with surrounding soils.” He began his study by applying measured loads to concrete, cast iron, and corrugated steel pipes buried under an embankment of 15 feet. His results indicated that the load on the concrete pipe was consistently 50% greater than the load on the corrugated steel pipe of approximately the same diameter (Figure 5-14). According to the study, “This load difference can be attributed to the difference in vertical deflection of the pipes that influenced the settlement ratios, and the magnitude of the shear stress components, as correctly theorized for flexible and rigid pipe materials”. (Rahman 2010)

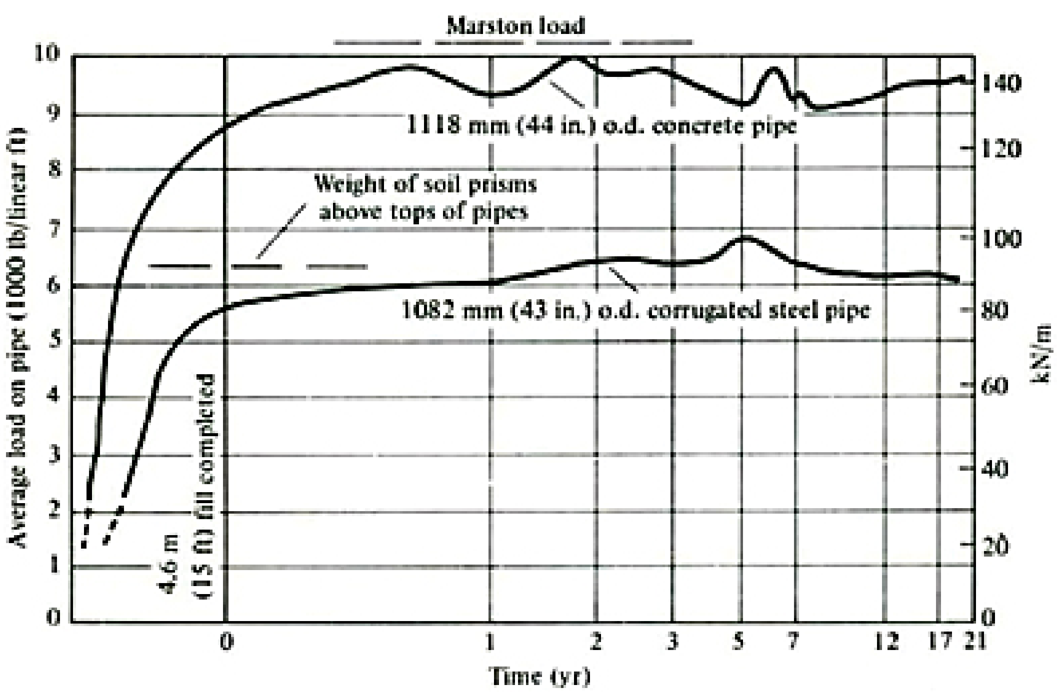



**Figure 5-14: Soil Land Differences in Rigid and Flexible Pipe (Rahman 2010)**

**5.5 Service Life**

America's aging infrastructure increases the risk of structural failures. Bridge corrosion and roadway degradation are extremely serious and increasing concerns. Culvert failures are no exception. Culvert failures have resulted in considerable damage to roadways causing widespread flooding and sinkholes throughout the United States. These failures are not only expensive but can be difficult to repair. Failures can occur without warning and thereby threatening the safety of American citizens. As a consequence, the FHWA and every state department of transportation stress the importance of considering the service life of the culvert in the selection process.

The Minnesota Department of Transportation's (MnDOT) Drainage Manual defines service life as "the period of service without a need for major repairs." "It is important to recognize that culverts are not assumed to be at or near the point of collapse at the end of their design service life. Rather, it is the period of little to no rehabilitative maintenance" (MnDOT 2000). Important factors that affect the service life of a culvert include:

- Corrosion
  - Soil Resistivity, Chloride, and Sulfate Concentration in Soil
  - Hydrogen Ion Concentration (pH) of the Surrounding Soil and Water

- Abrasion
  - Size, Shape, Hardness, and Volume of Bedload
  - Volume, Velocity, and Frequency of Streamflow

- Ultraviolet Radiation Exposure

- Flammability

### 5.5.1 Reinforced Concrete

The service life of a reinforced concrete culvert typically ends when the reinforcing steel has been exposed or significant cracks begin to form. The most critical factor affecting the service life of concrete pipe is chemical corrosion. Concrete can corrode when exposed to high concentrations of chloride and sulfate, as well as low pH and resistivity levels. Due to all of these varying components, many studies have been performed to evaluate the service life of concrete pipe including a study performed by the NCHRP entitled *Synthesis 474: Service Life of Culverts*. This study came at the request of the AASHTO subcommittee on culverts. The study collected the predication methods developed by various agencies and researchers to determine the expected service life of concrete pipe. Examples of a few of the prediction methods include the following (Maher et al. 2015):

- Utah DOT tests soil and water for resistivity, pH, soluble salts, and sulfate content, then uses charts to estimate the expected service life for various types of pipe. The expected service life of Portland cement concrete can be up to approximately 120 years.

- Arizona DOT assigns concrete pipe a service life of 100 years for installations where the pH is 5 or greater

- The U.S. Forest Service has defined acceptable conditions for concrete pipe to resist corrosion. If the pH of the water or soil surrounding the pipe is between 4.5 and 10 and the resistivity of the soil is greater than 1,500 ohm-cm, then the expected corrosion service life of concrete pipe is 75 years or greater.

- A study commissioned by the Ohio Department of Transportation found from a survey of 40 DOTs that service life of concrete culverts appeared to be limited to 70 to 80 years.

- A literature review by the National Research Council of Canada predicted the service life of concrete pipe varies from 50 to more than 100 years, depending on the environmental conditions to which the pipe is subjected.

- The Army Corp of Engineers recommends a design life of 70 to 100 years.

5.5.2 Corrugated Steel

The service life of a corrugated metal culvert ends when deterioration reaches the point of perforation. The most critical factors affecting the service life of steel pipe are corrosion, abrasion, and wall thickness. The most commonly used method to predict the durability of galvanized steel culverts is The California Method 643 published by the California Department of Transportation (CALTRANS). The California Method 643 considers two environmental factors when estimating the service life of steel culverts: the pH and electrical resistivity of the site and backfill materials. Applying these factors, CALTRANS developed a chart to estimate the maintenance-free service life of a galvanized steel culvert at any location. This chart is shown in Figure 5-15. (CALTRANS 2000)

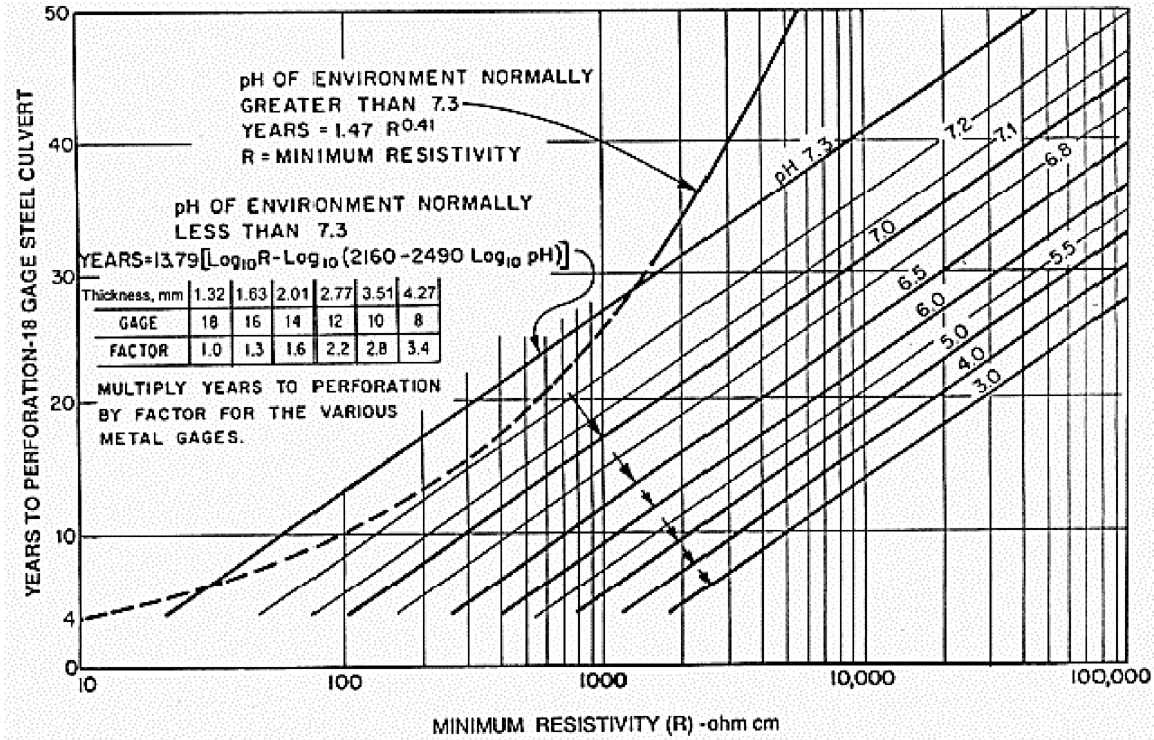



**Figure 5-15: Estimating Years to Perforation of Steel Culverts (CALTRANS 2000)**

Aluminized steel is more resistant to corrosion than galvanized steel. The University of Minnesota's report *Minnesota Steel Culvert Pipe Service-Life Map* concluded that "aluminized pipe overall provides a greater potential for higher service life than galvanized pipe" (Maher et al. 2015). According to NCHRP's Synthesis 474,

> *CALTRANS recommends using aluminized steel culverts instead of using other coatings or increasing the steel thickness in nonabrasive conditions with 5.5 < pH < 8.5 and minimum resistivity of at least 1,500 ohm-cm. With 5.5 < pH < 8.5 and resistivity less than 1,500 ohm-cm, CALTRANS does not recommend the use of aluminized type 2 steel culvert*. (Maher et al. 2015)

Recent testing has shown that polymer coated steel provides the most abrasion resistance as it can withstand Abrasion Level 3 conditions. The National Corrugated

Steel Pipe Association (NCSPA) guarantees a 100-year service life for polymer coated steel pipe if the environmental conditions shown in Table 5-17 are met.

**Table 5-17: Estimated Material Service Life for Corrugated Steel (NCSPA 2010)**

| Estimated Service Life | Site Environmental Conditions | Maximum FHWA Abrasion Level | Material |
|---|---|---|---|
| Minimum 100 Years | 5.0 < pH < 9.0<br>r > 1500 ohm-cm | Level 3 | Polymer Coated |
| | | Level 2 | Aluminized Type 2* |
| Minimum 75 Years | 4.0 < pH < 9.0<br>r > 750 ohm-cm | Level 3 | Polymer Coated |
| | 5.0 < pH < 9.0<br>r > 1500 ohm-cm | Level 2 | Aluminized Type 2 |
| Minimum 50 Years | 3.0 < pH < 12.0<br>r > 250 ohm-cm | Level 3 | Polymer Coated |
| Average 50 Years | 6.0 < pH < 10.0<br>2000 < r < 10,000 ohm-cm<br>> 50 ppm $CaCo_3$ | Level 2 | Galvanized |

*14 gauge minimum

### 5.5.3 Corrugated Aluminum

The most critical factors affecting the service life of aluminum pipe are corrosion, abrasion, and wall thickness. These factors are dependent on the pH and resistivity of the soil, as well as the velocity of the water flowing through the culvert. Therefore, the service life of aluminum varies by state and is typically deduced from a chart. The Maryland Department of Transportation (MDOT) published the chart shown below to estimate the service life of aluminum pipe. This chart is shown in Figure 5-16.

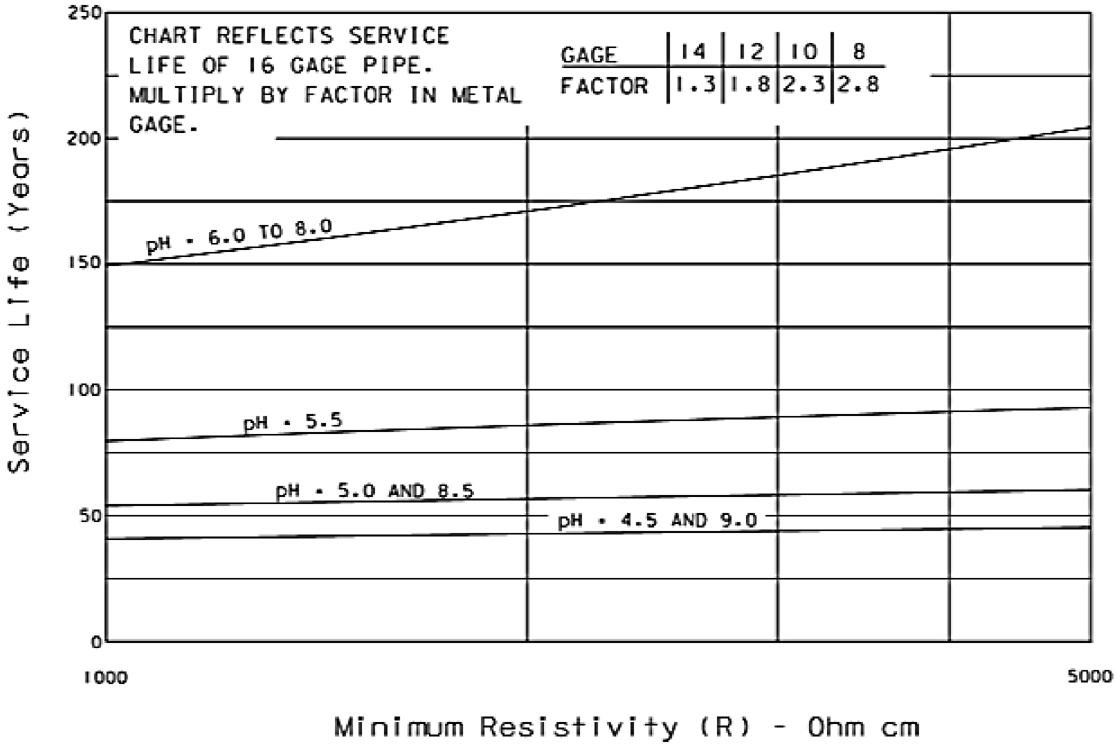



**Figure 5-16: Estimated Service Life vs. pH and Resistivity for Aluminum (MDOT 2009)**

The New York State Department of Transportation anticipates a 70-year service life for aluminum pipe, unless high velocities, potentially abrasive bed loads, or high concentrations of industrial waste are present (NYSDOT 2014). CALTRANS recommends a 50-year maintenance-free service life if the pH and resistivity of the soil, backfill, and drainage water meet the requirements stipulated in the Highway Design Manual (CALTRANS 2014).

As stated, the service life of metal pipe is dependent on wall thickness. The Oregon Department of Transportation's *Hydraulics Manual* specifies corrugated metal pipe wall thickness to the nearest 0.001 inch. Standard wall thicknesses and gage values used by the Oregon Department of Transportation are shown in Table 5-18. Most state transportation departments specify one standard service life for a pipe material.

However, some departments of transportation will increase the service life by a factor based on material type and wall thickness (Table 5-19).

**Table 5-18: Wall Thicknesses (ODOT 2014)**

| Galvanized Iron and Steel | | Aluminum | |
|---|---|---|---|
| Wall Thickness (inches) | Gage | Wall Thickness (inches) | Gage |
| 0.064 | 16 | 0.060 | 16 |
| 0.079 | 14 | 0.075 | 14 |
| 0.109 | 12 | 0.105 | 12 |
| 0.138 | 10 | 0.135 | 10 |
| 0.168 | 8 | 0.164 | 8 |

Note: Dimensions applicable to uncoated or metallic coated pipes.

**Table 5-19: Increase in Service Life of Aluminum and Steel Pipe per Wall Thickness (ODOT 2014)**

| Material | Wall Thickness (inches) | Material | Wall Thickness (inches) | Factor |
|---|---|---|---|---|
| Aluminum | 0.075 | Steel | 0.079 | 1.3 |
| Aluminum | 0.105 | Steel | 0.109 | 1.7 |
| Aluminum | 0.135 | Steel | 0.138 | 2.2 |
| Aluminum | 0.164 | Steel | 0.168 | 2.9 |

### 5.5.4 High Density Polyethylene

The most critical factors affecting the service life of high density polyethylene pipe are oxidation, ultraviolet radiation, and flammability. There is an extensive amount of research claiming that the service life of high density polyethylene pipe is well in excess of 100 years, even at deflections greater than 5%. The PPI references three published papers from independent studies on their website to support this claim. The papers were presented at Plastics Pipes XIII in Washington, DC and at the American Society of Civil Engineers Pipelines Conference in Chicago. The three published papers are as follows:

1. "Establishing 100-Year Service Life for Corrugated HDPE Drainage Pipe" by Michael Pluimer, Technical and Engineering Manager at Plastics Pipe Institute.

2. "Evaluate the Long-Term Stress Crack Resistance of Corrugated HDPE Pipes" by Y. Grace Hsuan, J-Y Zhang, and W-K Wong, of the Department of Civil, Architectural, and Environmental Engineering at Drexel University in Philadelphia, Pennsylvania.

3. "New Test Method to Determine Effect of Recycled Materials on Corrugated HDPE Pipe Performance as Projected by the Rate Process Method" by Dr. Gene Palermo, Palermo Plastics Pipe Consulting.

With the exception of Florida and Pennsylvania, state departments of transportation are hesitant to approve a 100-year service life for high density polyethylene pipe despite many of these studies. MDOT assumes a 75-year service life for corrugated polyethylene pipe. NYSDOT anticipates a 70-year service life for polyethylene pipe. According to *A Research Plan and Report on Factors Affecting Culvert Pipe Service Life in Minnesota*,

> *HDPE pipe has the durability and corrosive resistance to have a service life of over 100 years and is not significantly susceptible to freeze/thaw damage. We recommend adopting testing methods similar to the Florida testing methods for determining service life to identify HDPE pipes capable of yielding a 100-year service life*. (Taylor and Marr 2012)

In the paper "Establishing 100-Year Service Life for Corrugated HDPE Drainage Pipe," Michael Pluimer discusses the three failure modes of high density polyethylene. Pluimer begins the discussion by a brief abstract explaining the process to predict a long-term service life. First, Pluimer explains, the anticipated service conditions such as the environmental conditions, soil loads, traffic loads, and long-term stresses and strains must be assessed.  Secondly, the capacity of the material must be assessed.  The service conditions will vary by geographic location.  "While deep installations may result in large compressive stresses on the pipe, shallow installations are more subject to bending and tensile stresses" (Pluimer 2006).  Pluimer calculates the maximum demand placed on the pipe by the stress equation shown below.

$$\sigma = \frac{P}{A} \pm \frac{M\ c}{I} \qquad \text{(Equation 5-4)}$$

Where:

$\sigma$ = stress in pipe wall, psi
$P$ = hoop thrust in pipe wall, lb/in
$A$ = wall area, $in^2/in$
$M$ = moment in pipe wall, lb-in/in
$c$ = distance from extreme fiber in pipe wall to centroidal axis, in
$I$ = moment of inertia of pipe wall, $in^4/in$

The hoop stress is always compressive and increases as the cover height increases.  Because high density polyethylene is more prone to tensile failure than compressive failure, the hoop stress should be minimized and the bending stress maximized to determine the maximum tensile stress.  According to Pluimer, the worst-case condition for a high density polyethylene pipe is to have a shallow installation with high deflections, or to have a low hoop thrust with large bending stresses.  "It is interesting to note that if the pipe is properly installed, this type of condition should be rare as high deflections are typically not observed in shallow burials; such a condition is generally the result of poor installation practices" (Pluimer 2006).

Dr. Timothy McGrath for the Florida Department of Transportation determined the long-term stress and strain induced on the pipe wall using finite element analysis and theoretical AASHTO design calculations. Based on a total vertical deflection of 5% with minimum thrust, a long-term modulus of elasticity of 20,000 psi, and a factor of safety of 1.5, McGrath calculated a long-term stress of 450 psi and a long-term strain of 2.25%. However, McGrath's calculations only considered the circumferential stresses in the pipe. Citing two papers by Dr. Ian Moore, McGrath determined that the longitudinal stresses are of the same order of magnitude as the circumferential stresses. "Therefore, in order to ensure 100-year service life, the capacity of the material must be able to withstand this demand" (Pluimer 2006).

As mentioned previously, Pluimer discusses three failure modes of high density polyethylene crucial to the evaluation of the material's long-term performance.

- Stage I – Failures are ductile in nature, and occur at very high stress levels.
- Stage II – Failures are brittle types of fractures and occur at moderate stress levels. This is one of the primary failures modes and is associated with slow crack growth.
- Stage III – Failures occur as a result of chemical degradation.

Slow crack growth is a phenomenon characterized by crack propagation at low stress levels. ASTM F2136 *Standard Test Method for Notched, Constant Ligament-Stress (NCLS) Test to Determine Slow-Crack-Growth Resistance to HDPE Resins or HDPE Corrugated Pipe* compares the slow crack growth resistance for a limited set of resins. This test helps prevent Stage II failures. Pluimer also recommends utilizing the Rate Process Method to determine the 100-year service life of high density polyethylene. The Rate Process Method "takes advantage of the Arrhenius principle of time-temperature superposition to accelerate the test and extrapolate data to predict service life at the anticipated service temperature" (Pluimer 2006). The Rate Process Method is included in ASTM D2837 *Standard Test Method for Obtaining Hydrostatic Design Basis for Thermoplastic Pipe Materials or Pressure Design Basis for Thermoplastic Pipe Products*. The Rate Process equation shown below relates time and hoop stress as a function of absolute temperature. A pipe's brittle failure performance is determined by the three coefficients. "Once the three coefficients are determined, the Rate Process equation is used to determine if the time to failure at the required service conditions is greater than 100 years" (Pluimer 2006).

$$\log_t = A + \frac{B}{T} + \frac{C \log S}{T} \qquad \text{(Equation 5-5)}$$

Where:

| | | |
|---|---|---|
| *t* | = | time, hour |
| *S* | = | hoop stress, psi |
| *T* | = | absolute temperature, °K |
| *A, B, C* | = | constants |

Another test method to evaluate high density polyethylene's resistance to Stage II failures was proposed by Dr. Grace Hsuan for the Florida Department of Transportation. Hsuan's test focused on the junction between the corrugation and liner. A diagram of the junction specimen is shown in Figure 5-17. This junction was chosen based on Hsuan and McGrath's prior work on NCHRP Report 429 *HDPE Pipe: Recommended Material Specifications and Design Requirements*. A field sample of high density polyethylene pipe with noted slow crack growth failures is shown in Figure 5-18. "By the nature of the geometry of this junction, it will act as a stress concentration point where slow crack growth failure is most likely to occur. This proposed junction test consists of a tensile load applied to the test specimen while immersed in a water bath" (Hsuan and McGrath 2005, Pluimer 2006).

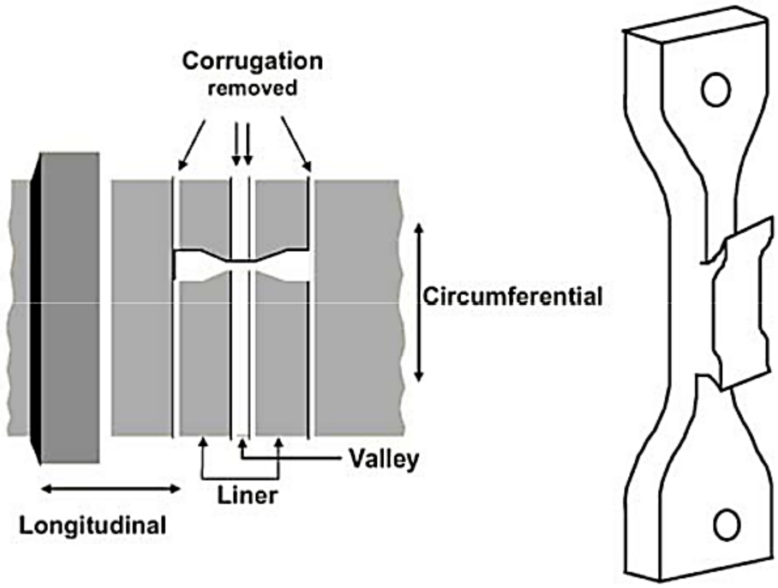



**Figure 5-17: Diagram of Junction Specimen (Hsuan and McGrath 2005)**

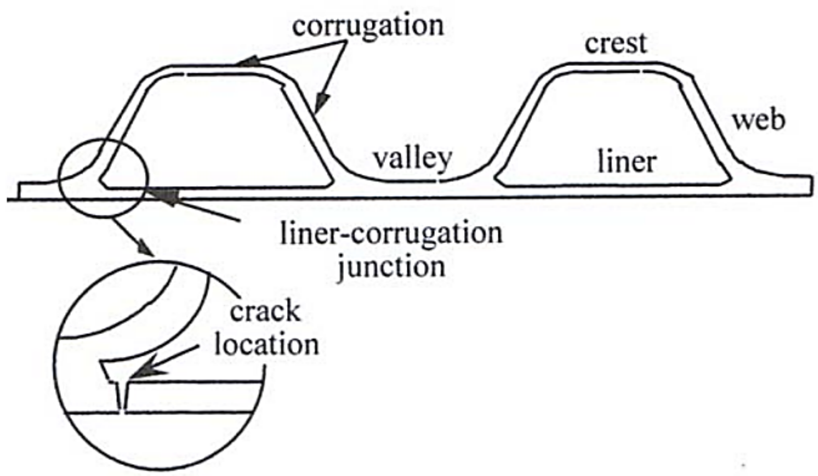



**Figure 5-18: Location of Linear-Corrugation Junction (Hsuan and McGrath 2005)**

Stage III failures are prevented by the addition of antioxidants to the material formulation. Antioxidants protect the resin from oxidative degradation. "Thus, if it can be shown that there will be some antioxidant present in the material over the 100 year service life, one can be assured that the pipe will not experience Stage III failures in this time period" (Pluimer 2006). Figure 5-19 illustrates how poor oxidation stabilizers can affect the service life of the pipe. A lack of antioxidants shift the Stage III failure curve to the left. If the Stage III failure curve is shifted far enough, the service life will be detrimentally impacted.

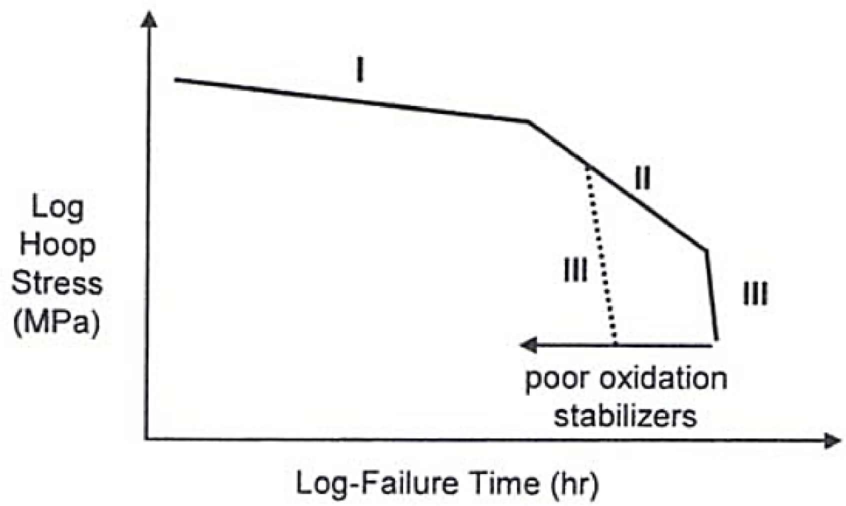



**Figure 5-19: Antioxidants Effect on Service Life (Hsuan and McGrath 2005)**

Pluimer recognizes two tests to determine the antioxidant activity in a polyethylene formulation. The first test is known as the induction temperature test or the thermal stability test. This test is performed "by heating a test specimen at a constant rate and recording the temperature at which oxidation initiates." The second test is known as the oxidation induction time test. This test is performed by "measuring the time to achieve oxidation at a given test temperature." (Pluimer 2006)

In conclusion, Pluimer reaffirms that the 100-year service life is dependent on the installation conditions to determine the demand placed on the pipe and the material properties of the pipe. McGrath determined that the maximum factored tensile stress and strain in a pipe was 450 psi and 2.25%, respectively, based on a total vertical deflection of 5%, a long-term modulus of elasticity of 20,000 psi, and a factor of safety of 1.5. In order to determine the material's capability to meet these predetermined demands, Hsuan applied the Rate Process Method to predict Stage II performance. He performed an oxidation induction time test to check the antioxidant performance. This ensured Stage III failures would not occur. Based on these calculations and tests, Pluimer concludes that high density polyethylene corrugated pipe can be evaluated for a 100 year service life.

### 5.5.5 Polyvinyl Chloride

Walker (1981) reported a 2-year study into the effects of UV aging on mechanical properties of PVC pipe. It was found that after two years of exposure under some of the worst aging conditions in North America the modulus of tensile elasticity and tensile strength of PVC pipe was unchanged. This is evidence that PVC pipe's ability to resist external soil loads and traffic loads has not been adversely altered by two years of direct sunlight exposure. The impact strength of the pipes, however, was found to have decreased by 20.3% over the two years. Walker contends that even the lowest impact strengths reported during this evaluation should not concern PVC pipe consumers or impair PVC pipe's performance. The report concluded that the desirable mechanical properties of PVC pipe, formulated for buried use, were not adversely affected to a significant extent by two full years of outdoor weathering and direct exposure to sunlight.

In 2014, Dr. Steven Folkman of Utah State University published the report *PVC Pipe Longevity: Affordability and The 100+ Year Benchmark Standard*. The objective of the research was to explore the reliability and longevity of polyvinyl chloride pipe through a comprehensive review of pipe excavation studies. The report stemmed from a study performed by Utah State University's Buried Structures Laboratory in 2012. The study sought to analyze the statistical results of water main breaks in the United States and Canada. "The findings of the research demonstrated that PVC pipe has the lowest rate of main breaks of pipe materials considered in the study, which included ductile iron, cast iron, steel, concrete, and asbestos cement" (Folkman 2014). Major findings of Dr. Folkman's research are as follows:

- PVC pressure pipes exhumed after being in operation for almost 30 years have not suffered any loss of strength. All tested pipes would be expected to exceed a 100-year life under normal operations conditions.
- After over 35 years of operation, PVC pipes have virtually no change in mechanical properties due to ageing.
- Based on stress regression, slow crack growth and fatigue testing, the service life of PVC pressure pipe should exceed 100 years.
- Recently excavated PVC pipes, some nearly 50 years old, were tested by Utah State University and met all applicable standards. They are expected to easily exceed 100 years of service life.

In addition to excavation reports in the United States, Dr. Folkman also included excavation studies from Australia, Europe, and Canada. In Australia, polyvinyl chloride pressure pipes were exhumed after 25 years of operation and subjected to testing based on the Australian Standards. The pipes were installed in a variety of terrains traversing both roads and rail lines. However, the pressure class of the pipe was never upgraded to accommodate the varying dynamic loads. Results concluded that "… there has been no degradation in the strength or elongation characteristics of the PVC during the service life of the pipes. The exhumed pipes have not suffered any loss of strength as a consequence of operating under pressure for almost 30 years" (Folkman 2014).

FDOT allows polyvinyl chloride pipes to have a 100-year service life if the requirements of ASTM F949 are met. Polyvinyl chloride pipes not meeting the requirements of ASTM F949 must have a 50-year service life. CALTRANS's specifies a minimum design service life for culverts on all projects regardless of material type. The minimum design service life per CALTRANS is shown in Table 5-20. In Revision 87 of NYSDOT's *Highway Design Manual*, only steel, reinforced concrete, aluminum, polyethylene, and polypropylene are listed as acceptable culvert materials. However, NYSDOT also specifies a minimum design service life by location. The minimum design service life per NYSDOT is shown in Table 5-21.

**Table 5-20: Design Service Life for Culverts (CALTRANS 2014)**

| Roadbed Widths Greater than 28 feet | Greater than 10 feet of Cover | Roadbed Widths 28 feet or less and with less than 10 feet of Cover | Installations under Interim Alignment |
|---|---|---|---|
| 50 years | 50 years | 25 years | 25 years |

**Table 5-21: Design Service Life for Pipe Culverts by Location (NYSDOT 1996)**

| Location | Design Life (Years) |
|---|---|
| Driveways | 20 |
| Significant Locations[1] | 70 |
| Other Locations | 50 |

[1] Significant locations are defined as follows:

a. Highways functionally classified as Interstates and Other Freeways
b. Natural watercourses, or channels, such as perennial streams
c. Under high fills – 15 feet or greater
d. Locations with high traffic volumes
e. Locations where long off-site detours would be required if the culver failed

In 2012, the Delaware Department of Transportation (DelDOT) issued a Design Guidance Memorandum to provide guidance on current policy for the selection of pipe material. Service life would now be assigned in accordance with the corresponding pipe installation level. These design service lives are shown in Table 5-22.

**Table 5-22: Design Service Life based on Installations (DelDOT 2012)**

| Installation Level | Roadway Classification | Service Life | Pipe Material |
|---|---|---|---|
| I | Expressway, Arterial or Collector | 75+ | Reinforced Concrete, High Density Polyethylene |
| II | Local Roadways, High Volume Commercial Entrances (i.e., Shopping Centers) | 50+ | Corrugated Aluminum, Spiral Rib Aluminum, Polyvinyl Chloride, Polypropylene |
| III | Commercial Entrances, Multi-family Residential Entrances | 25+ | Polymer-Coated Corrugated Steel, Aluminum-Coated Type 2 Corrugated Steel |
| IV | Residential Driveway Entrances | 15+ | Galvanized Corrugated Steel |

Section II.2.1.1 "Pipe Material Selection Policy for Cross Drains, Side Drains, and Storm Drains" of LaDOTD's *Engineering Directives and Standards Manual*, establishes the design life and allowable material types used for cross drains, side drains, and storm drains. Interestingly, LaDOTD only allows reinforced concrete pipes, reinforced concrete pipe arches, and ribbed polyvinyl chloride pipes to be used as cross drain beneath freeways, urban, rural and suburban arterials, and urban and rural collectors having four lanes. Table 5-23 shows the design service life and approved material type based on application. Table 5-24 lists all of the referenced acronyms.

**Table 5-23: Design Service Life and Material Selection for Culverts (LaDOTD 2005)**

| Application | Design Service Life | Joint Type | Materials |
|---|---|---|---|
| Storm Drain Pipes<br>Flumes Other<br>Watertight Systems | 70 years | T3 | RCP(A), RPVCP |
| Storm Drain Pipe<br>(Outfall) | 50 years | T3 | BCCSP(A), CAP(A),<br>CSP(A), RPVCP |
| Cross Drain Pipes for:<br>Free Ways<br>Urban Arterial<br>Rural Arterial<br>Urban Collector (4 lanes)<br>Rural Collector (4 lanes)<br>Suburban Arterial | 70 years | T3 | RCP(A), RPVCP |
| Cross Drain Pipes for:<br>Urban Collector (2 lanes)<br>Rural Collector (2 lanes)<br>Urban Local<br>Rural Local<br>Suburban Collector | 50 years | T2 | RCP(A), BCCSP(A), CAP(A),<br>RPVCP, CPEPDW[1] |

| | | | |
|---|---|---|---|
| Side Drain | 30 years | T1 | RCP(A), BCCSP(A), CAP(A), CSP(A), RPVCP, CPEPDW |
| Side Drain (Erosion) | 30 years | T1 | BCCSP(A), CAP(A), CSP(A), RPVCP, CPEPDW |
| Side Drain (Bridge Drains) | 50 years | T1 | BCCSP(A), CAP(A), CSP(A), RPVCP, CPEPDW |

[1] CPEPDW applicable on roadways where traffic volume does not exceed 3000 ADT

**Table 5-24: Material Type Abbreviations and Definitions (LaDOTD 2005)**

| Abbreviation | Definition | Abbreviation | Definition |
|---|---|---|---|
| RCP | Reinforced Concrete Pipe | RCP(A) | Reinforced Concrete Pipe Arch |
| CAP | Corrugated Aluminum Pipe | CAP(A) | Corrugated Aluminum Pipe Arch |
| CSP | Corrugated Steel Pipe | CSP(A) | Corrugated Steel Pipe Arch |
| BCCSP | Bituminous Coated Corrugated Steel Pipe | BCCSP(A) | Bituminous Coated Corrugated Steel Pipe Arch |
| RPVCP | Ribbed Polyvinyl Chloride Pipe | CPEPDW | Corrugated Polyethylene Pipe Double Wall |

### 5.5.6 Polypropylene

The most critical factors affecting the service life of polypropylene pipe are oxidation, ultraviolet radiation, and flammability. In 2014, ADS released the news article "ADS HP Polypropylene Pipe Meets Specification for 100-Year Design Life in Side Drain, Cross Drain and Storm Sewer Applications." The article announced that the Florida Department of Transportation (FDOT) approved the use of ADS' High Performance polypropylene pipes ranging from 12-inch to 60-inch in diameter for 100-year service life applications. According to FDOT documents,

> *Polypropylene pipe has passed the needed testing to be accepted for 100-year side drain, cross drain and storm sewer applications. Until project plans and specifications reflect this update, PP pipe may be selected by the contractor for any project where high-density polyethylene (HDPE) pipe is allowed*. (ADS 2014)

While polypropylene has recently emerged as a culvert material in North America, it has been used in Europe for 40 years. In 2015, The European Plastic Pipes and Fittings Association published a technical report on the service life of non-pressure polyethylene and polypropylene pipes. According to *100 Year Service Life of Polypropylene and Polyethylene Gravity Sewer Pipes*, polyolefin products are expected to have a service life of at least 100 years. The report was published because "no scientific study on service life expectancy had been done on pipes that operate with a constant strain in sewage and drainage applications" (TEPPFA 2014). As stated in the news article *Study: Service Life for PE, PP Sewer Pipes at least 100 Years,*

> *The findings were based on a two-year study of pipes excavated from five sites in Finland, Norway, Denmark and Germany. One pipe made of first-generation high density PE had been in the ground 38 years and the PP pipes had been in operation 10-23 years. The tests found no excessive deterioration or degradation and the results demonstrate the long-term performance of solid wall and structured wall sewer pipes using long-term, real-time data*. (Kavanaugh 2015)

### 5.6 Durability

Durability is crucial to a culvert's serviceability. As defined by the PPI, "durability is the property to resist erosion, material degradation and subsequent loss of function due to environmental or other service conditions" (PPI 2008). Material degradation can occur by cracking, tearing, spalling, abrading, or corroding. It can also occur due to joint separation, excessive buckling, and deflection. The corrosion potential of a site is dependent on the soil resistivity and the chloride and sulfate concentration in the soil, as well as the hydrogen ion concentration in the soil and water. High density polyethylene, polyvinyl chloride and polypropylene are unaffected by corrosion, resistivity, chlorides, sulfates, and pH levels.

### 5.6.1 Chemical Corrosion

Each day 850 water mains break in the United States and Canada. One in four of those breaks are caused by corrosion. As mandated by the United States Congress, the FHWA released a study in 2002 on the direct costs associated with corrosion. According to the study "Corrosion Costs and Preventative Strategies in the United States", corrosion costs the United States' water and wastewater systems over $50.7 billion annually. The American Society for Civil Engineers estimated that 2.6 trillion gallons of potable water are lost every year through leaking pipes or 17% of all water pumped in the United States. (Koch et al. 2002)

The most common reason for a culvert to fail is due to a gradual weakening caused by corrosion (Figure 5-20). Corrosion can occur on both the inside and outside of the culvert. According to Victor Chaker's (1989) *Effects of Soil Characteristics on Corrosion: Issue 1013*, the rate of deterioration is a function of many factors. This includes properties of the pipe and its protective coatings, the nature of the soil, and the chemicals in solution in the soil water. The presence of soils and waters in the pipes containing acids, alkalis, dissolved salts and organic industrial wastes is a likely indicator of corrosion. Many factors carry these contaminants including surface water, ground water, sanitary effluent, acid rain, marine environments and mine drainage.

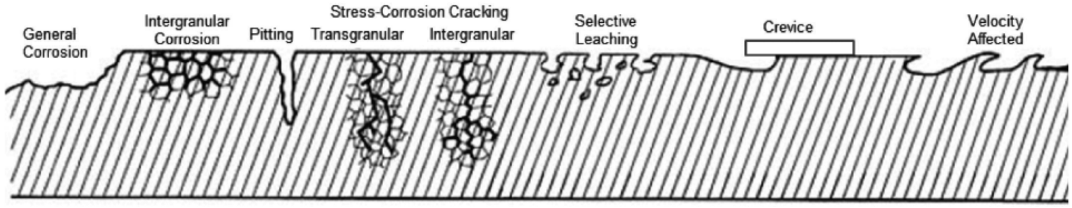



**Figure 5-20: Schematic of Common Corrosion Mechanisms (Maher et al. 2015)**

Corrosion most commonly attacks unprotected metal culverts and the reinforcement in concrete pipes. Corrosion of the reinforcing steel or other embedded metals is the leading cause of deterioration in concrete. When the reinforcing steel corrodes, the resulting rust occupies a greater volume than the reinforcing steel. An example of this expansion is shown in Figure 5-21. According to the article *Types and Causes of Concrete Deterioration*, “This expansion creates tensile stresses in the concrete, which can eventually cause cracking, delamination, and spalling” (PCA 2002). The article goes on to state:

> *Steel corrodes because it is not a naturally occurring material. Steel, like most metals except gold and platinum, is thermodynamically unstable under normal atmospheric conditions and will release energy and revert back to its natural state – iron oxide, or rust. This process is called corrosion*. (PCA 2002)

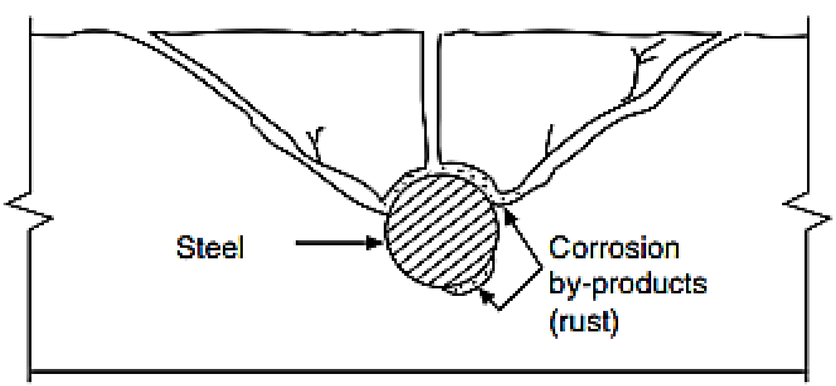



**Figure 5-21: Expansion of Corroding Steel in Concrete (PCA 2002)**

An example of a corroded concrete box culvert is shown in Figure 5-22. The box culvert was installed north of Walsenburg in Colorado. Soil samples were taken from the site which indicated an extremely high sulfate concentration. Three samples had a sulfate concentration of 16,800 ppm, 11,200 ppm, and 20,800 ppm. An example of a corroded corrugated metal culvert is shown in Figure 5-23. The metal culvert had corroded so severely that the bottom of the culvert had completely disintegrated.

**Figure 5-22: Corrosion of Reinforced Concrete Culvert (Molinas and Mommandi 2009)**

**Figure 5-23: Corrosion of Corrugated Metal Culvert (Matthews et al. 2012)**

The alkaline environment of concrete provides steel with corrosion protection. According to the article *Types and Causes of Concrete Deterioration*, “At the high pH, a thin oxide layer forms on the steel and prevents metal atoms from dissolving. Without the passive film, the steel would corrode at rates at least 1,000 times higher” (PCA 2002). Despite concrete’s inherent protection, corrosion can occur when the passivating layer is destroyed. While this thin oxide layer will not stop corrosion, it will reduce the corrosion rate to an insignificant level. The destruction of the passivating layer occurs “when the alkalinity of the concrete is reduced or when the chloride concentration in concrete is increased to a certain level” (PCA 2002). A list of chemicals known to deteriorate concrete is shown in Table 5-25. Table 5-26 lists likely causes of culvert deterioration.

**Table 5-25: Chemicals that Deteriorate Concrete (PCA 2002)**

### Chemicals That Deteriorate Concrete

***Promote rapid deterioration of concrete:***

Aluminum Chloride
Calcium Bisulfite
Hydrochloric Acid (all concentrations)*
Hydrofluoric Acid (all concentrations)
Nitric Acid (all concentrations)
Sulfuric Acid, 10-80 percent*
Sulfurous Acid

***Promote moderate deterioration of concrete:***

Aluminum Sulfate*
Ammonium Bisulfate
Ammonium Nitrate
Ammonium Sulfate*
Ammonium Sulfide
Ammonium Sulfite
Ammonium Superphosphate
Ammonium Thiosulfate
Castor Oil
Cocoa Bean Oil*
Cocoa Butter*
Coconut Oil*
Cottonseed Oil*
Fish Liquor[1]
Mustard Oil*
Perchloric Acid, 10%
Potassium Dichromate
Potassium Hydroxide (>25%)
Rapeseed Oil*
Slaughterhouse Waste[2]
Sodium Bisulfate
Sodium Bisulfite
Sodium Hydroxide (>20%)
Sulfite Liquor
Sulfuric Acid, 80% Oleum*
Tanning Liquor (if acid)
Zinc Refining Solutions[3]

* Sometimes used in food processing or as food or beverage ingredient. Ask for advisory opinion of Food and Drug Administration regarding coatings for use with food ingredients.

[1] Contains carbonic acid, fish oils, hydrogen sulfide, methyl amine, brine, other potentially active materials

[2] May contain various mixtures of blood, fats and oils, bile and other digestive juices, partially digested vegetable matter, urine, and manure, with varying amounts of water.

[3] Usually contains zinc sulfate in sulfuric acid. Sulfuric acid concentration may be low (about 6 percent in "low current density" process) or higher (about 22-28% in "high current density" process).

**Table 5-26: Likely Causes of Culvert Deterioration (Wagener and Leagjeld 2014)**

| Observed Condition | Likely Cause |
|---|---|
| Invert and crown cracking width in excess of 0.10" in RCP culverts | • Dead and live loading on culvert exceeding design capacity<br>• Increased loading on culvert due to increased soil or groundwater |

| | |
|---|---|
| | elevations |
| Slabbing (slabs of concrete "peeling" away from the sides of the pipe and a straightening of the reinforcement due to excessive deflection or shear cracks) in RCP culverts | • Dead and live loading on culvert exceeding culvert design capacity<br>• Increased loading on culvert due to increased soil or groundwater excavations<br>• Improper bedding of culvert |
| Deflections exceeding 7% in flexible culverts | • Dead and live loading on culvert exceeding culvert design capacity<br>• Increased loading on culvert due to increased soil or groundwater excavations<br>• Improper installation or selection of haunching materials or insufficient compaction<br>• Loss of soil through pipe wall or joints<br>• Piping of materials on exterior of culvert<br>• Excessive construction equipment loading with insufficient cover |
| Loss/erosion of invert | • Erosion of culvert material due to stream bed loading (all pipe materials)<br>• Corrosion or deterioration of culvert material due to pH of water, resistivity of soil, chemical attack, etc. (all pipe materials)<br>• Corrosion of reinforcement and resulting expansive forces resulting in delamination of concrete (RCP)<br>• Freeze-thaw deterioration (RCP) |
| Joint separation and infiltration of soil | • Improperly seating of joint during installation<br>• Movement of pipe due to slope erosion, free-thaw or settlement<br>• Movement of pipe due to excessive deflection or structural deterioration<br>• Buoyancy of culvert with insufficient cover |

Plastic pipes are unaffected by most inorganic acids, alkalis, and aqueous solutions. Dow Chemical released a case-study praising the superiority of plastics used in nuclear power plants. The case-study, *The Power of Plastic,* cited that one of the main advantages to replacing carbon steel in safety-related pipe systems with high density polyethylene was the plastic's inability to corrode. "Polyethylene material does not corrode, rust, rot, pit, tuberculate or support biological growth, and it has an outstanding field performance record (for more than half a century) in water piping systems" (Dow 2009). Prior to the installation at Callaway Nuclear Power Plant, high density polyethylene pipes had never been used for a safety-related American Society of Mechanical Engineers Class 3 water pipe application at a nuclear power plant in North America.

AmerenUE, a subsidiary of Ameren Corporation and owner of Callaway Nuclear Power Plant cited several advantages to using high density polyethylene for safety-related pipe at nuclear power plants (Dow Chemical Company 2009):

> 1. HDPE pipe is leak-free when produced and installed properly, even at joints, which can be as strong and leak-free as the pipe itself through use of the heat fusion joining technique.
>
> 2. It offers seismic resistance, in that it can safely accommodate repetitive pressure surges above its static pressure rating and is well-suited for seismic loading due to its natural flexibility.
>
> 3. HDPE is easier and more cost-efficient to install than carbon steel

However, the high density polyethylene pipes studied by Dow Company were installed in a pristine environment that was free of soil and water contamination and climatic influence. Though unlikely, some concentrated acids and oxidizing agents can pose a threat to plastic pipe at extremely elevated temperatures.

PVC, like HDPE, is also a non-conductor so there are no galvanic or electrochemical effects in PVC pipe (Uni-Bell 2013). PVC also suffers no damage from attack of normal or corrosive soils and is not affected by sulfuric acid in the concentrations found in sewer systems (Uni-Bell 2013).

According to Gabriel and Moran (1998), state DOTs that have reported using plastic as an alternative material for drainage pipes have noted that these pipes are highly resistant to the various corrosive agents, sulfates, chlorides, and other aggressive salts found in soil and highway drainage effluents. The Federal Lands Highway Division of FHWA policy states that plastic alternatives may be specified without regard to resistivity and pH of the site with regard to corrosion (Gabriel and Moran 1998).

#### *5.6.1.1 Chloride Concentration*

Chloride ions are the most extensively documented contaminant that cause corrosion of the embedded steel in concrete. The embedded steel is more susceptible to corrosion if the concrete cover is inadequate, cracked, or highly permeable. As stated in the Portland Cement Association's (PCA) article *Types and Causes of Concrete Deterioration*,

> *When the chloride content at the surface of the steel exceeds a certain limit, called the threshold value, corrosion will occur if water and oxygen are also available. Federal Highway Administration studies found that a threshold limit of 0.20% total chloride by weight of cement could induce corrosion of reinforcing steel in bridge decks*. (PCA 2002)

Table 5-27 presents the maximum chloride ion content associated with various types of concrete members. Chloride ions are present in deicing salts and seawater. The degradation is often accelerated in regions where successive freeze-thaw cycles occur. Figure 5-24 illustrates the frequency of freeze-thaw exposure in the United States. Based on the figure, the majority of Alabama rarely experiences freeze-thaw exposure. Only the northern part of the state may experience an occasional freeze-thaw exposure. However, higher elevations receive a greater frequency of exposure.

**Table 5-27: Maximum Chloride Ion Content of Concrete (ACI 318 2014)**

| Type of Member | Maximum $Cl^-$* |
|---|---|
| Prestressed concrete | 0.06 |
| Reinforced concrete exposed to chloride in service | 0.15 |
| Reinforced concrete that will be dry or protected from moisture in service | 1.00 |
| Other reinforced concrete construction | 0.30 |

*Water-soluble chloride, percent by weight of cement

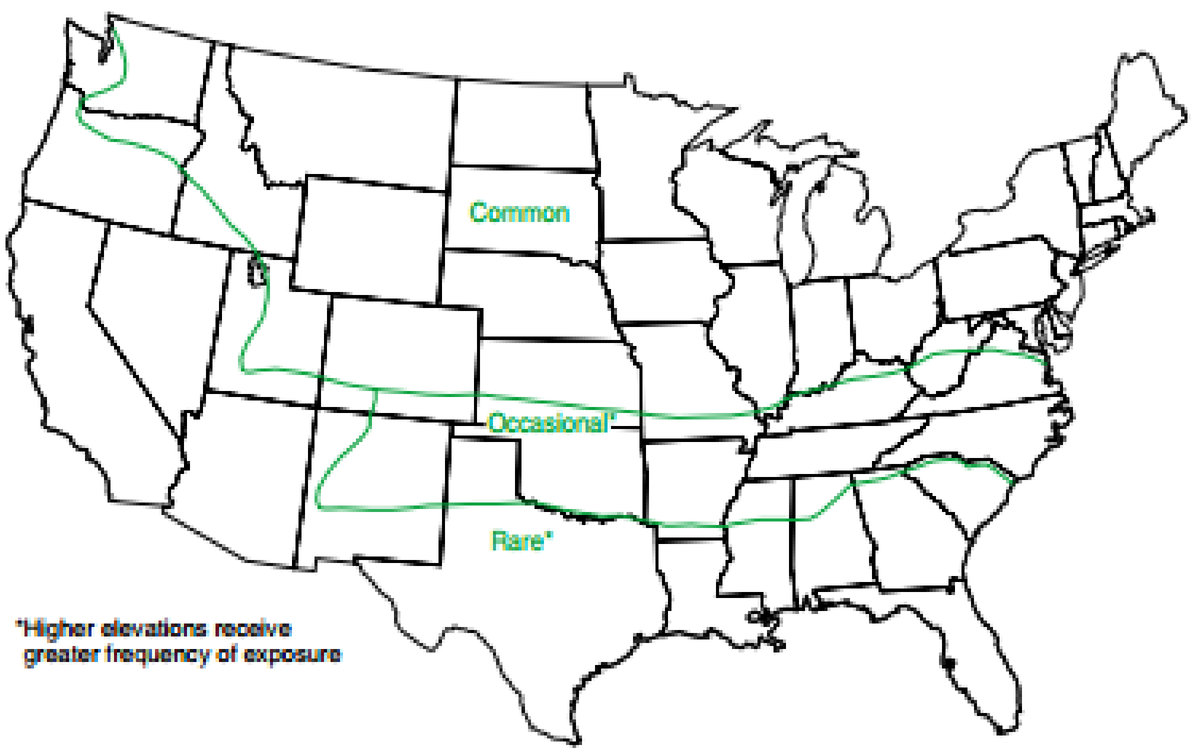



**Figure 5-24: Frequency of Freeze-Thaw Exposure in the United States (PCA 2002)**

The AASHTO LRFD Bridge Construction Specifications specifies a maximum in-place width of 0.100-inch for noncorrosive conditions and 0.010-inch for corrosive conditions (AASHTO 2009). According to NCHRP *Synthesis 474*, "The general view was that in the case of very narrow cracks, the process of concrete leachate interacting with atmospheric or waterborne $CO_2$ would cause calcite and other carbonate deposits that would seal such cracks" (Maher et al. 2015). This process known as autogenous healing prompted FDOT to initiate a study at South Florida University (Figure 5-25). Results of the study determined that:

> *Significant autogenous healing was not detected in cracks as narrow as 0.020 in. Corrosion tests showed that significant reinforcement corrosion took place in a short period of time with 0.100-in.-wide cracks, but that corrosion damage was much slower with cracks 0.020 in. wide. Allowable crack width limits above 0.100 in. are not acceptable under any circumstances*. (Maher et al. 2015)

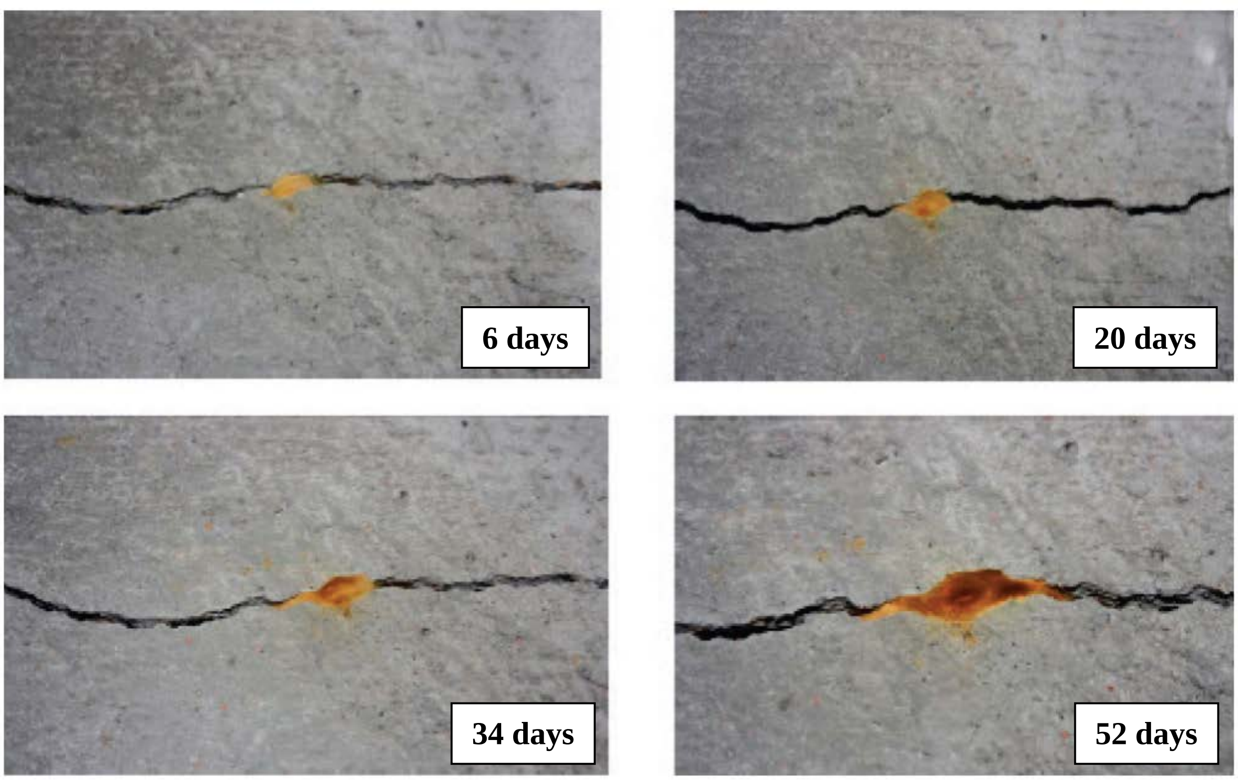



**Figure 5-25: Chloride Attack on Reinforced Concrete Pipe (Maher et al. 2015)**

*5.6.1.2 Sulfate Concentration*

While a high concentration of sulfates may corrode metal culverts, they are typically more damaging to concrete. According to *Types and Causes of Concrete Deterioration*, "Sulfates can attack concrete by reacting with hydrated compounds in the hardened cement. These reactions can induce sufficient pressure to disrupt the cement paste, resulting in loss of cohesion and strength" (PCA 2002). A sulfate attack is greatly influenced by environmental conditions. An attack is more common in dry areas such as the Northern Great Plains and parts of the Western United States.

The Colorado Department of Transportation (CDOT) sponsored Colorado State University to research the relationship between the service life of a culvert and various parameters including the pH level and chloride and sulfate concentrations levels in the surrounding soil and water. The report *Development of New Corrosion/ Abrasion Guidelines for Selection of Culvert Pipe Materials* indicated that problems arise for concrete pipes when the sulfate concentration exceeds 1,000 parts per million (ppm). (Molinas and Mommandi 2009)

The South Carolina Department of Transportation (SCDOT) and MnDOT both stipulate that concrete pipes are sufficient to withstand sulfate concentrations of 1,000 ppm or less. CALTRANS considers a site corrosive if the sulfate concentration exceeds 2,000 ppm. FDOT does not consider concrete vulnerable to accelerated deterioration unless the sulfate concentration exceeds 5,000 ppm. However, chloride ions are considered to be a larger threat in Florida as sulfate concentrations rarely exceed 1,500 ppm.

The most efficient way to protect against a sulfate attack is to choose a cement with a limited amount of tricalcium aluminate. ASTM C150, *Standard Specification for Portland Cement*, covers ten types of Portland cement: Type I, Type IA, Type II, Type IIA, Type II (MH), Type II (MH)A, Type III, Type IIIA, Type IV, and Type V. Type II or Type V cement is recommended when sulfate resistance is desired (Table 5-28). Other resistance factors may include reducing the water-to-cement ratio, using a higher strength concrete, or applying special coatings.

In 2003, the Montana Department of Transportation sponsored a nationwide survey to determine the service life guidelines developed by other state departments of transportation. All 50 States were encouraged to participate, however only 20 States responded. According to the responses, “Two of the twenty states limit the use of reinforced concrete pipe based on sulfates. Eighteen states do not” (Molinas and Mommandi 2009). Table 5-29 depicts various sulfate exposures and the recommended cement type.

**Table 5-28: Optional Composition Requirements (ASTM C150)**

| Cement Type | Applicable Test Method | I and IA | II and IIA | II(MH) and II(MH)A | III and IIIA | IV | V | Remarks |
|---|---|---|---|---|---|---|---|---|
| Tricalcium aluminate ($C_3A$)[B] , max, % | See Annex A1 | ... | ... | ... | 8 | ... | ... | for moderate sulfate resistance |
| Tricalcium aluminate ($C_3A$)[B] , max, % | See Annex A1 | ... | ... | ... | 5 | ... | ... | for high sulfate resistance |
| Equivalent alkalies ($Na_2O + 0.658K_2O$), max, % | C114 | 0.60[C] | 0.60[C] | 0.60[C] | 0.60[C] | 0.60[C] | 0.60[C] | low-alkali cement |

[A] These optional requirements apply only when specifically requested. Verify availability before ordering. See Note 2.
[B] See Annex A1 for calculation.
[C] Specify this limit when the cement is to be used in concrete with aggregates that are potentially reactive and no other provisions have been made to protect the concrete from deleteriously reactive aggregates. Refer to Specification C33 for information on potential reactivity of aggregates.

**Table 5-29: Various Sulfate Concentrations (PCA 2002)**

| Sulfate exposure | Water-soluble sulfate ($SO_4$) in soil, percent by mass | Sulfate ($SO_4$) in water, ppm | Cement type[2] | Maximum water-cementitious material ratio, by mass | Minimum design compressive strength, MPa (psi) |
|---|---|---|---|---|---|
| Negligible | Less than 0.10 | Less than 150 | No special type required | — | — |
| Moderate[1] | 0.10 to 0.20 | 150 to 1500 | II, MS, IP(MS), IS(MS), P(MS) I(PM)(MS), I(SM)(MS) | 0.50 | 28 (4000) |
| Severe | 0.20 to 2.00 | 1500 to 10,000 | V, HS | 0.45 | 31 (4500) |
| Very severe | Over 2.00 | Over 10,000 | V, HS | 0.40 | 35 (5000) |

[1] Seawater.
[2] Pozzolans or slags that have been determined by test or service record to improve sulfate resistance may also be used.
Test method: *Method for Determining the Quantity of Soluble Sulfate in Solid (Soil or Rock) and Water Samples,* Bureau of Reclamation, 1977.
Source: Adapted from Bureau of Reclamation 1981 and ACI 318.

*5.6.1.3 Electrochemical Corrosion*

Resistivity is a measure of the soil's ability to conduct electrical current. Resistivity is measured in units of ohm-centimeters and, it greatly affects metal culverts. "Unlike zinc that acts as a sacrificial barrier, an aluminum coating serves as a long-lasting barrier. Aluminum reduces the potential differences between cathodes and anodes and therefore decreases the rate of the electrochemical process" (Gabriel and Moran 1998; French 2013). According to MnDOT's *Drainage Manual*, "The greater the resistivity of the soil, the less capable the soil is of conducting electricity and the lower the corrosive potential" (MnDOT 2000). As stated in the *Drainage Manual,*

> *Resistivity values in excess of about 5,000 ohm-cm are considered to present limited corrosion potential. Resistivities below the range of 1,000 to 3,000 ohm-cm will usually require some level of pipe protection, depending on the corresponding pH level*. (MnDOT 2000)

According to FDOT'S *Drainage Handbook Optional Pipe Material*s (FDOT 2014), resistivity values greater than 3,000 ohm-cm are considered high and will impede corrosion. Resistivity values less than 1,000 ohm-cm will accelerate corrosion. Typical soil corrosion potential resistivity values are shown in Tables 5-30 and 5-31. Table 5-32 lists typical resistivity values associated with soil and water.

**Table 5-30: Typical Soil Corrosion Potential Resistivity Values based upon AISI (1999) (Maher et al. 2015)**

| Soil Corrosion Potential | Resistivity (Ohm-Centimeter) |
|---|---|
| Normal | $R > 2,000$ |
| Mildly Corrosive | $2,000 > R > 1,5000$ |
| Corrosive | $1,500 > R$ |

**Table 5-31: Typical Soil Corrosion Potential Resistivity Values based upon Gabriel and Moran 1998 (Maher et al. 2015)**

| Soil Corrosion Potential | Resistivity (Ohm-Centimeter) |
|---|---|
| Negligible | $R$ > 10,000 |
| Very Low | 10,000 > $R$ > 6,000 |
| Low | 6,000 > $R$ > 4,500 |
| Moderate | 4,500 > $R$ > 2,000 |
| Severe | 2,000 > $R$ |

**Table 5-32: Typical Resistivity Values (Molinas and Mommandi 2009)**

| Soil | | Water | |
|---|---|---|---|
| **Classification** | **Ohm-Centimeter** | **Source** | **Ohm-Centimeter** |
| Clay | 750 – 2,000 | Seawater | 25 |
| Loam | 3,000 – 10,000 | Brackish | 2,000 |
| Gravel | 10,000 – 30,000 | Drinking Water | 4,000 + |
| Sand | 30,000 – 50,000 | Surface Water | 5,000 + |
| Rock | 50,000 – Infinity | Distilled Water | Infinity |

The type of soil in which a culvert is buried is critical as granular soil exhibits a higher resistivity than non-granular soil. This soil-resistivity relationship is shown in Table 5-33. Agronomy and Soils Professor Charles C. Mitchell, Junior, of Auburn University issued the report *Soils of Alabama* (Mitchell 2008) in furtherance for the United States Department of Agriculture. In the report, Professor Mitchell classifies Alabama's soil into seven major areas around the state. Figure 5-26 presents Professor Mitchell's seven classifications.

**Table 5-33: Typical Corrosion Potential of Various Soil Conditions (Maher et al. 2015)**

| Soil Type | Description of Soil | Aeration or Drainage | Water Table |
|---|---|---|---|
| Lightly Corrosive | - Sands or sandy loams<br>- Light-textured silt loams<br>- Porous loams or clay loams thoroughly oxidized to great depths | Good | Very low |
| Moderately Corrosive | - Sandy loams<br>- Silt loams<br>- Clay loams | Fair | Low |
| Badly Corrosive | - Clay loams<br>- Clays | Poor | 2 to 3-feet below surface |
| Unusually Corrosive | - Muck<br>- Peat<br>- Tidal marsh<br>- Clays and organic soils | Very Poor | At surface or extreme impermeability |

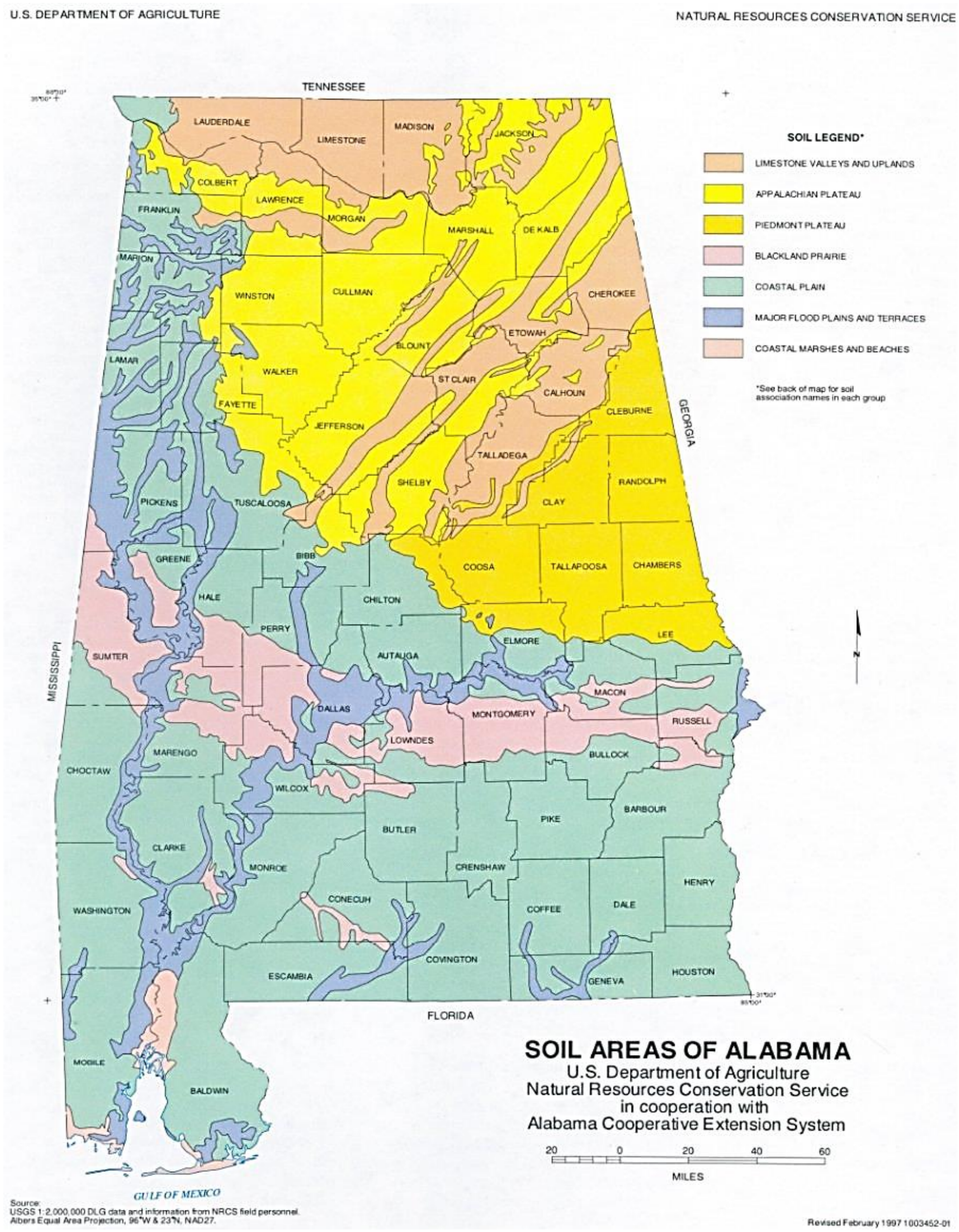



**Figure 5-26: Soil Areas of Alabama (Mitchell 2008)**

### *5.6.1.4 Hydrogen Ion Concentration*

As defined by the United States Environmental Protection Agency (EPA n.d.), pH is an expression of hydrogen ion concentration in water. Specifically, pH is the negative logarithm of hydrogen ion ($H^+$) concentration in an aqueous solution:

$$pH = -\log_{10}\left(H^{+}\right) \quad \text{(Equation 5-6)}$$

The term is used to indicate the degree of basicity or acidity of a solution ranked on a scale of 0 to 14, with pH 7 being neutral. As the concentration of $H^+$ ions in solution increases, acidity increases and pH gets lower, below 7. When pH is above 7, the solution is basic.

The hydrogen ion scale shown in Figure 5-27 distinguishes between a basic and acidic solution. According to FDOT's *Drainage Handbook Optional Pipe Materials,* "When a culvert is placed in an environment in which the pH is too low (≤ 5.0) or too high (≥ 9.0), the protective layers of the culvert (concrete, galvanizing, aluminizing, etc.) can weaken, leaving the metal vulnerable to early corrosion" (FDOT 2014). It is extremely important that the appropriate culvert material is chosen for the specific environmental conditions of the site. One of the most common preventative measures is to apply a protective coating.

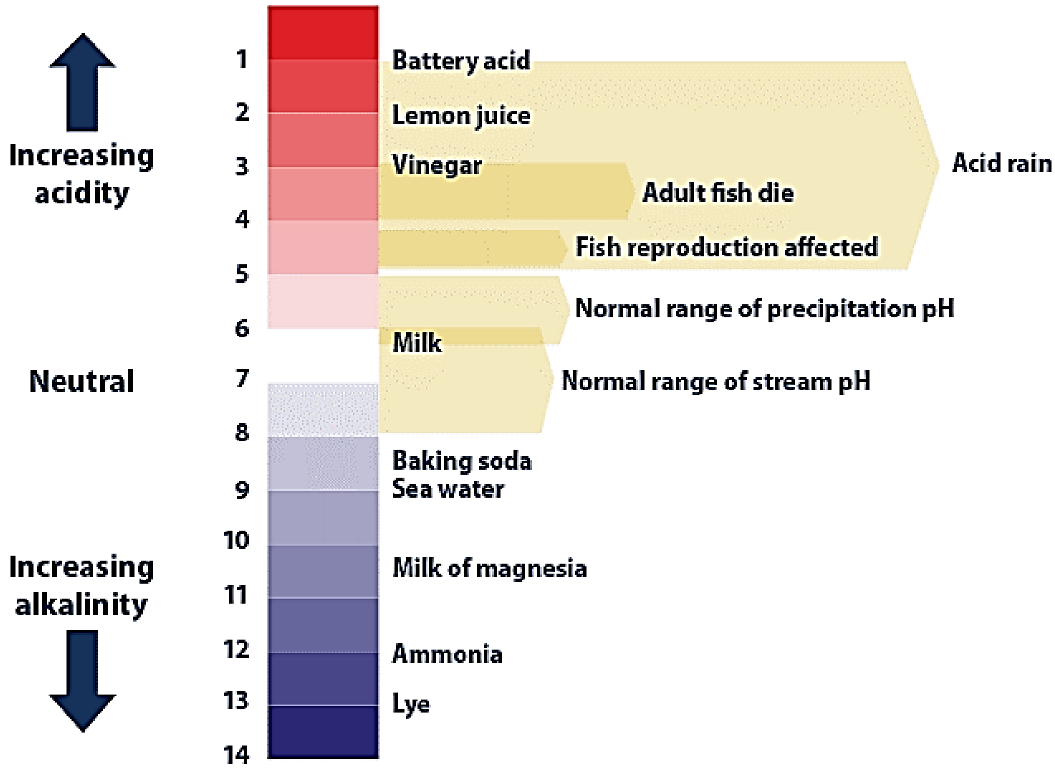



**Figure 5-27: Hydrogen Ion Scale (EPA n.d.)**

The NCSPA published the *CSP Durability Guide* to provide environmental ranges for corrugated steel pipe products. An excerpt from this guide is shown in Figure 5-28. According to the *Guide*, in natural enviroments galvanized steel corrodes slower than steel. Galvanized steel should not be used where the pH is outside the range of 6.0 to 12.5. Aluminumized steel is quite stable in neutral solutions, as well as many acid solutions. However, aluminized steel is vulnerable to alkalies and should not be used where the pH is greater 9.0. Based on the pH levels, aluminumized steel has an advantage over galvanized steel in lower pH environments. (NCSPA 2000)

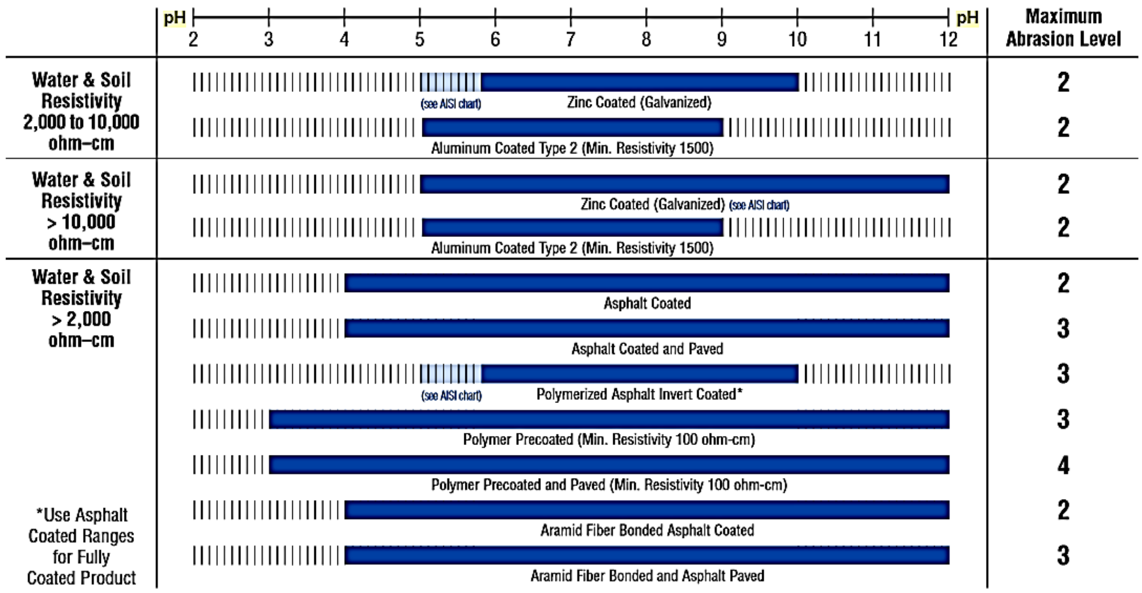



**Figure 5-28: Environmental Guidelines for Corrugated Steel Pipe (NCSPA 2000)**

Plastic pipe culverts to be used without regard to the resistivity and pH of the soil. Table 5-34 depicts various environmental ranges based on pH and resistivity. However, the same liberties cannot be applied to metal culverts or concrete culverts. According to the FHWA's *Hydraulic Design of Highway Culverts*, "Metal culverts are adversely affected by acidic and alkaline conditions in the soil and water, and by high electrical conductivity of the soil. Concrete culverts are sensitive to saltwater environments and to soils containing sulfates and carbonates" (Schall et al. 2012). The hydrogen ion range for various culvert materials is shown in Table 5-35. Table 5-36 illustrates the Corps of Engineers' environmental range for metal pipes.

**Table 5-34: Environmental Ranges (NCSPA 2000)**

| Environmental Ranges | pH | Resistivity |
|---|---|---|
| Normal Conditions | 5.8 – 8.0 | > 2,000 ohm-cm |
| Mildly Corrosive | 5.0 – 5.8 | 1,500 – 2,000 ohm-cm |
| Corrosive | < 5.0 | < 1,500 ohm-cm |

**Table 5-35: Hydrogen Ion Range (Schall et al. 2012)**

| Type of Material Used to Make Pipe | pH Range |
|---|---|
| Galvanized Steel | 6.0 < pH < 10 |
| Aluminum | 4.0 < pH < 9.0 |
| Reinforced Concrete | < 5.0 |

**Table 5-36: Corps of Engineers' Range for Metal Pipe (French 2013; Corps of Engineers 1998)**

| Type of Material Used to Make Pipe | Soil and Water pH | Minimum Soil Resistivity (ohm-cm) |
|---|---|---|
| Galvanized Steel | 6.0 – 8.0 | ≥ 2,500 |
| Aluminized Steel, Type 2 | 5.0 – 9.0 | ≥ 1,500 |
| Aluminum | 5.0 – 9.0 | ≥ 1,500 |
| Stainless Steel, Type AISI 409 | 5.0 – 9.0 | ≥ 1,500 |

### 5.6.2 Abrasion

As defined by the NCHRP, "Abrasion is the progressive loss of section or coating of a culvert by the continuous, rapid movement of turbulent water containing a bedload of particulate matter" (Maher et al. 2015). Abrasion often accelerates corrosion by stripping away the surface material or protective coating of a culvert. Once the protective coating has been removed, the culvert's main defense against corrosion has been destroyed (Figure 5-29). The combined effects of corrosion and abrasion are well documented in NCHRP's *Synthesis 474*:

> *The abrasive properties of bedload that is traveling at high velocities and is harder than the exposed pipe invert or coating will erode metal, concrete, and thermoplastic pipes. When corrosion and abrasion operate together in this manner, they can produce a larger detrimental effect than either would if applied in isolation. Abrasion accelerates corrosion by*

*removing protective coatings, and corrosion can produce products less resistant to abrasion*. (Maher et al. 2015)

**Figure 5-29: Corrosion Accelerated by Abrasion of Metal Culvert (Maher et al. 2015)**

Steel pipe is the most susceptible to abrasion. However, aluminum pipe offers no improvement. According to *Synthesis 474*, "Although aluminum culverts are occasionally specified to combat corrosion, plain aluminum is typically not recommended for abrasive environments since tests indicate that aluminum can abrade as much as three times faster than the rate of steel" (Maher et al. 2015). While the California Design Information Bulletin 83-2003 considers aluminized steel equivalent to galvanized steel in abrasive resistance, the NCSPA recommends using non-metallic coatings over metallic coatings for increased abrasion resistance (Figure 5-30).

○ Shaded Circles Indicate Applicable Coatings

★ See AISI Chart

| COATING | WATERSIDE | | | | | | |
|---|---|---|---|---|---|---|---|
| | Normal Conditions | Mildly Corrosive | Corrosive | Non-Abrasive/Low Abrasion (Lvl. 1 & 2) | Moderate Abrasion (Level 3) | High Abrasion (Level 4) | Provides Additional Soil Side Protection |
| Zinc Coated (Galvanized) | ★ | ★ | ○ | ● | ○ | ○ | ○ |
| Aluminum Coated Type 2 | ● | ● | ○ | ● | ○ | ○ | ○ |
| Asphalt Coated | ● | ● | ● | ● | ○ | ○ | ● |
| Asphalt Coated and Paved | ● | ● | ● | ● | ● | ○ | ● |
| Polymerized Asphalt Invert Coated* | ● | ● | ○ | ● | ● | ○ | ○ |
| Polymer Precoated | ● | ● | ● | ● | ● | ○ | ● |
| Polymer Precoated and Paved | ● | ● | ● | ● | ● | ● | ● |
| Polymer Precoated w/ Polymerized Asphalt | ● | ● | ● | ● | ● | ● | ● |
| Aramid Fiber Bonded Asphalt Coated | ● | ● | ● | ● | ○ | ○ | ● |
| Aramid Fiber Bonded and Asphalt Paved | ● | ● | ● | ● | ● | ○ | ● |
| High Strength Concrete Lined | ● | ● | ○ | ● | ● | ○ | ○ |
| Concrete Paved Invert (75mm (3") Cover) | ● | ● | ○ | ● | ● | ● | ○ |

* Use Asphalt Coated Environmental Ranges for Fully Coated Product

Note: Coatings listed under additional soil side protection are generally considered to provide 100 years service life from a soil side perspective within appropriate environmental conditions.[1]

**Figure 5-30: Product Usage Guidelines for Corrugated Steel Pipe (NCSPA 2000)**

Abrasion is dependent on the velocity of water. As the velocity increases, the sand, gravel or stones carried by the water more forcefully attack the inside of a culvert. The Federal Lands Highway Division of the FHWA developed four levels of abrasion to help characterize the abrasion potential of a site (Table 5-37). According to *Synthesis 474*, "Generally, flow velocities less than 5 feet per second (ft/s) are not considered to be abrasive, even if bedload material is present. Velocities that exceed 15 ft/s and carry a bedload are considered to be very abrasive" (Maher et al. 2015).

**Table 5-37: Abrasion Levels (Gabriel and Moran 1998)**

| | | |
|---|---|---|
| Non-Abrasive | Level 1 | Non-abrasive conditions exist in areas of no bed load and very low velocities. This is the condition assumed for the soil side of drainage pipes |
| Low Abrasion | Level 2 | Low abrasive conditions exist in areas of minor bed loads of sand and velocities of 5 feet/second or less |
| Moderate Abrasion | Level 3 | Moderate abrasive conditions exists in areas of moderate bed loads of sand and gravel and velocities between 5 feet/second and 15 feet/second |
| Severe Abrasion | Level 4 | Severe abrasive conditions exist in areas of heavy bed loads of sand, gravel and rock and velocities exceeding 15 feet/second |

Plastic pipe exhibits good abrasion resistance and will likely not experience the dual action of corrosion and abrasion. "Multiple tests and field evaluations prove that it takes significantly longer to abrade through high density polyethylene pipe walls than through concrete and metal" (PPI 2008; French 2013). However, this claim was based on tests using small aggregate sizes flowing at low velocities. The effects of large bedload particles or high velocity flows are not well documented. In addition, rehabilitative strategies have not been specifically developed for plastic pipe due to their more recent emergence as a culvert material. While invert paving is a very common strategy for metal culverts, it would be "ineffective with plastic pipes because of their smooth surface and inability to achieve a satisfactory bond" (Maher et al. 2015). Corrosion and abrasion guidelines followed by the New Mexico Department of Transportation are shown in Table 5-38.

**Table 5-38: New Mexico DOT Corrosion and Abrasion Guidelines (Molinas and Mommandi 2009)**

| Material | Recommended Adjustments for Abrasion | | | |
|---|---|---|---|---|
| | **Low Abrasion Level 1** | **Mild Abrasion Level 2** | **Moderate Abrasion Level 3** | **Severe Abrasion Level 4** |
| Concrete Pipe | No Addition | No Addition | No Addition | Modify Mix Design |
| Aluminized Steel Type II | No Addition | No Addition | Add One Gage | Add One Gage and Pave Invert |
| Galvanized Steel (2 and 3 oz. Coating) | No Addition | Add One Gage* | Add Two Gages* | N/A |
| Polymer Pre-coated Galvanized Steel | No Addition | No Addition | Add One Gage | Add One Gage and Pave Invert |
| Aramid Fiber Bonded Galvanized Steel | No Addition | No Addition | No Addition | Add One Gage |
| Aluminum Alloy | No Addition | No Addition | Add One Gage | Add One Gage and Pave Invert |
| Thermoplastic Pipe (PVC and HDPE) | No Addition | No Addition | No Addition | N/A |

*A field applied concrete paved invert per ASTM A849 may be substituted for one (1) gage thickness

In 2007, Glenn DeCou and Paul Davies published the report *Evaluation of Abrasion Resistance of Pipe and Pipe Lining Materials*. The objective of the report was to evaluate the abrasion resistance of various pipe materials in a real-world, non-laboratory setting over a 5-year period. The various pipe materials were installed in the Sierra foothills at the Shady Creek crossing of Highway 49 in Nevada County, Northern California. The test site is known to be abrasive and recovering from major hydraulic gold mining activities that occurred in the mid-1800s to the early 1900s. The pipe and pipe liner materials evaluated in the project are shown in Table 5-39.

**Table 5-39: Pipe and Liner Materials Evaluated at Shady Creek (Davies and DeCou 2007)**

| Pipe or Pipe Lining Material | Supplier | Specification Reference |
|---|---|---|
| 4" Concrete over Galvanized Corrugated Steel Pipe | Pacific Corrugated | AASHTO M 36 |
| Composite Steel Spiral Rib Pipe | Pacific Corrugated | AASHTO M 36 |
| Steel Spiral Rib Pipe (Ribs at 7 ½" Pitch) | Pacific Corrugated | AASHTO M 36 |
| Aluminum Spiral Rib Pipe (Ribs at 7 ½" Pitch) | Pacific Corrugated | AASHTO M 36 |
| Steel Spiral Rib Pipe with Polymerized Asphalt | Pacific Corrugated | AASHTO M 36 |
| Galvanized Corrugated Steel Pipe – 2 ⅔ x ½ Std. Corrugation | Pacific Corrugated | AASHTO M 36 |
| Corrugated Aluminized Steel Pipe, Type 2 – 2 ⅔ x ½ Std. Corrugation | Pacific Corrugated | AASHTO M 36 |
| Galvanized Corrugated Steel Pipe – 2 ⅔ x ½ Std. Corrugation with Polymeric Coating | Pacific Corrugated | AASHTO M 36 |
| Galvanized Corrugated Steel Pipe – 2 ⅔ x ½ Std. Corrugation with Polymeric Coating and Polymerized Asphalt | Pacific Corrugated | AASHTO M 36 |
| Galvanized Corrugated Steel Pipe – 2 ⅔ x ½ Std. Corrugation Bituminous Coated and Paved | Pacific Corrugated | AASHTO M 36 |
| Aluminum Spiral Rib Pipe (Ribs at 7 ½" Pitch) | Contech | AASHTO M 196 |
| Corrugated High Density Polyethylene | ADS | AASHTO M 294 |
| Ribbed Polyvinyl Chloride | J-M Pipe | AASHTO M 304 |
| Cured-in-Place Pipe – Polyester Resin | Insituform | ASTM F 1216 and F 1743 |

| | | |
|---|---|---|
| Reinforced Concrete Pipe | California Concrete Pipe Association | AASHTO M 170 |
| Basalt Tile | Abresist | |
| Calcium Aluminate Mortar | Lafarge | Various ASTM |

Table 5-40 documents the pH and minimum resistivity values of the soil and water used for design. Average flow velocities of 12 to 18 feet per second were recorded for the site. Bedload particles included medium grain sizes between 3 and 11 mm. Interestingly, results of the study concluded that high density polyethylene experienced more abrasive wear than polyvinyl chloride when exposed to abrasive conditions. Additional conclusions formed by Glenn DeCou and Paul Davies are as follows:

- Abrasion wear to pipes, liners and linings in the field is not linear with time. It is event driven and dependent on the number and size of events during any given year.
- Most of the coated steel pipe outperformed non-coated steel pipe.
- Smoother profiles evidenced less abrasive wear than did corrugated profiles.
- All of the pipe materials tested evidenced significantly less abrasive wear than did concrete pipes.

Glenn DeCou and Paul Davies made reference to a separate study conducted at the California State University at Sacramento in 1989. The study *Abrasion Resistance of Polyethylene and Other Pipes* (Gabriel 1989) was performed in a laboratory using four-foot-long test sections mounted on a rocking table with a gradation of approximately two inches. Bedload particles consisted of crushed quartz gravel. The average test velocity was 3 feet per second. While most of the testing was performed at a pH level of 7.0, some testing occurred at pH levels of 4.0 and 5.5. Pipe materials that were tested included high density polyethylene, polyvinyl chloride, corrugated steel, corrugated aluminum, and concrete. Results of the study are summarized in Figures 5-31 and 5-32.

**Table 5-40: Tested pH and Resistivity Values at Shady Creek (Davies and DeCou 2007)**

| pH | | Minimum Resistivity, Ohm-Cm | |
|---|---|---|---|
| **Soil** | **Water** | **Soil** | **Water** |
| 5.8 | 6.8 | 10,300 | 15,100 |
| 6.0 | 6.9 | 8,300 | 14,700 |

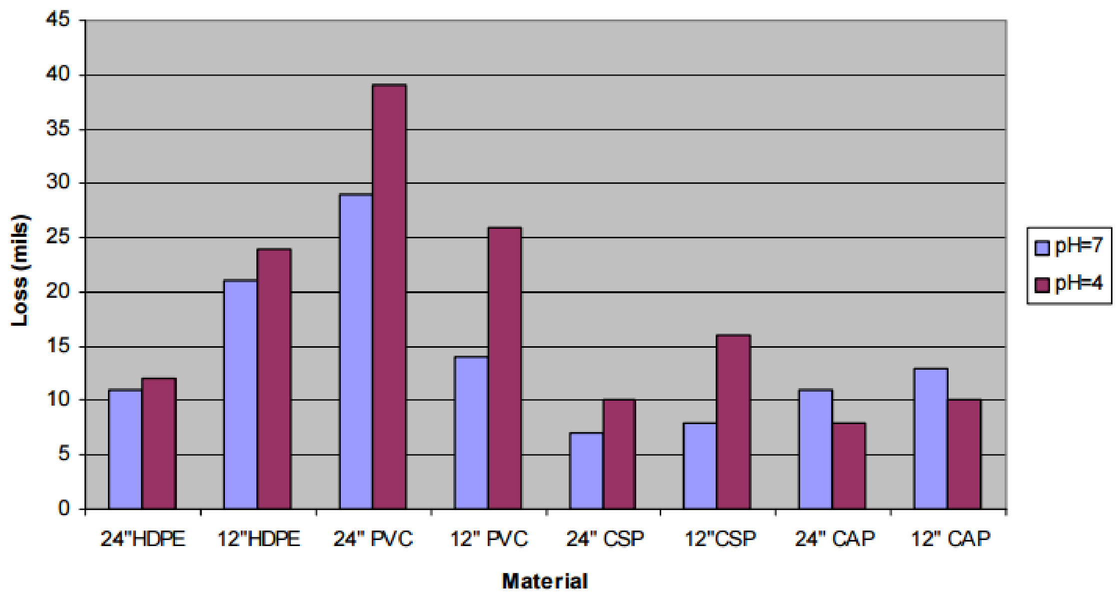



**Figure 5-31: Abrasion Study Test Results for Flexible Pipe (Davies and DeCou 2007)**

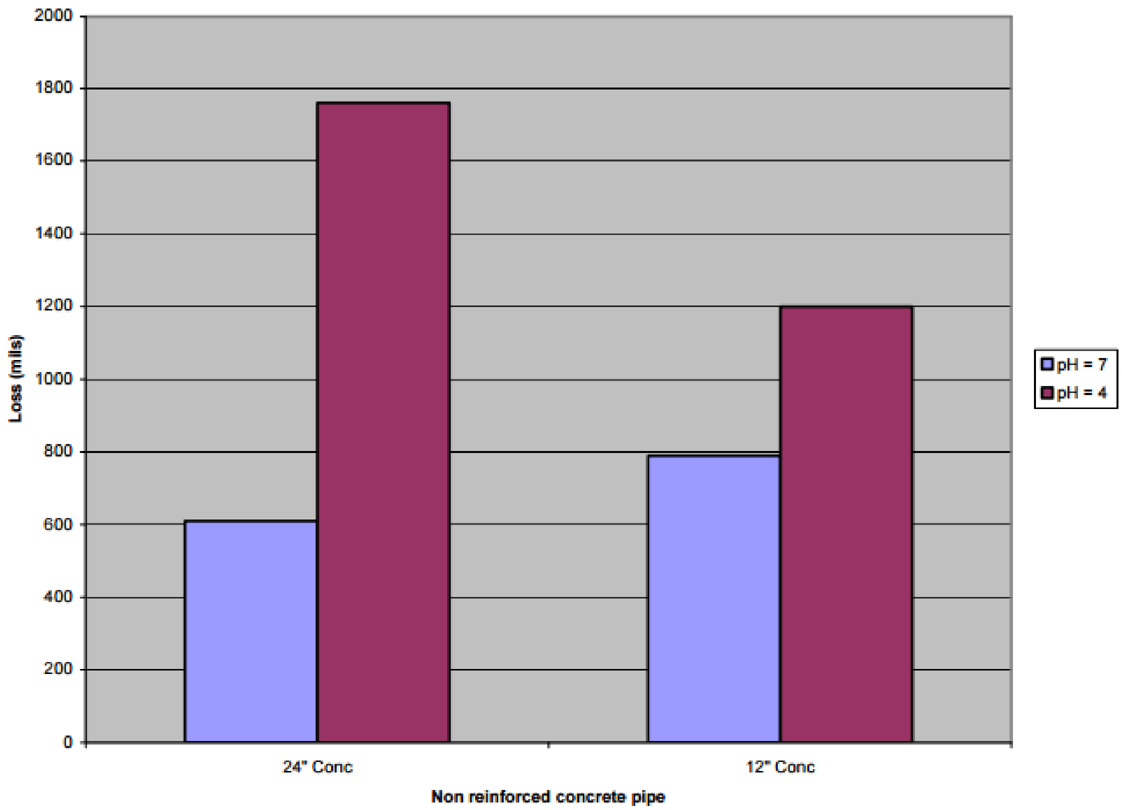



**Figure 5-32: Abrasion Study Test Results for Rigid Pipe (Davies and DeCou 2007)**

As shown in the figures, large diameter polyvinyl chloride pipes and non-reinforced concrete pipes experienced the greatest abrasive wear regardless of the pH level. Additional conclusions presented in *Abrasion Resistance of Polyethylene and Other Pipes* include:

- High density polyethylene, polyvinyl chloride, corrugated steel and corrugated aluminum evidenced less abrasive wear in both neutral and acid environments than did concrete pipes.

- Polyvinyl chloride, concrete and corrugated steel experienced greater abrasive wear in an acid environment than did these same pipe materials in a neutral environment.

- Corrugated aluminum experienced less abrasive wear in an acid environment than did these same pipe materials in a neutral environment

### 5.6.3 Ultraviolet Radiation

Plastic pipes are affected by oxidation and ultraviolet radiation. According to the report *Evaluation of Polypropylene Drainage Pipe* published by the Virginia Center for Transportation Innovation and Research, “Polypropylene is highly susceptible to oxidation and undergoes oxidation more readily than polyethylene. Polypropylene can begin to disintegrate to an oxidized powder right after formation if no antioxidants are added during manufacturing” (Hoppe 2011).

Ultraviolet stabilizers and antioxidant packages are used to protect plastic pipes against this form of degradation. ASTM D3350 *Standard Specification for Polyethylene Plastics Pipes and Fittings Materials* requires a minimum carbon black content of 2.0% be used for all polyethylene compounds. According to ADS’s *Drainage Handbook*:

> *With the UV stabilizers incorporated into polyethylene and polypropylene, the radiation can only penetrate a thin layer into the pipe wall over the service life of the pipe. It is important to understand that once the outer layer has been faded by the sun, it functions as a shield to protect the rest of the pipe from further degradation. A high percentage of the pipe’s original strength properties remain intact because the majority of the wall remains unharmed.* (ADS 2014)

The NCHRP published the report *HDPE Pipe: Recommended Material Specifications and Design Requirements* which describes the use of carbon black as a combatant to ultraviolet radiation. The report states, “Carbon black is added to the resin formulation to provide ultraviolet resistance. The pipe is only vulnerable to ultraviolet resistance light during the storage period and before backfilling. Once the pipe is covered by soil, it is not subjected to ultraviolet light” (Hsuan Y. G. and McGrath 1999).

While polyethylene pipes are black due to the carbon black resin, polyethylene pipes are grey. Carbon black is not used on polypropylene. Instead, ADS incorporates an outdoor, weather-able pigment system plus a Hindered Amine Light Stabilizer (HALS) on polypropylene products. BASF, the North American affiliate of BASF SE, is a producer of HALS and discusses its use in the article *Get a Grip on Light with Uvinul®!*. According to the article,

> *In contrast to the physically active UV absorbers, the various HALS react chemically. They interrupt the propagation of polymer degradation by scavenging the radicals created at chain breaks, thus rendering them harmless. Their high level of protection is due to the fact that each stabilizer molecule is not only able to react once but may react many times. This sustainably decelerates the chain reaction of degradation*. (BASF n.d.)

M. S. Jones of the Building Research Association of New Zealand published the paper *Effects of UV Radiation on Building Materials*. The paper examined the effects of ultraviolet radiation on polymer-based products as well as the use of accelerated weathering techniques. *Effects of UV Radiation on Building Materials* warns of the serious effects of photo-degradation including discoloration, micro-cracking and embrittlement of substrates (Figure 5-33). "These effects [micro-cracking and embrittlement] are often accompanied by extensive deterioration in the mechanical properties of the materials, such as tensile strength, impact strength and elongation" (Jones 2002).

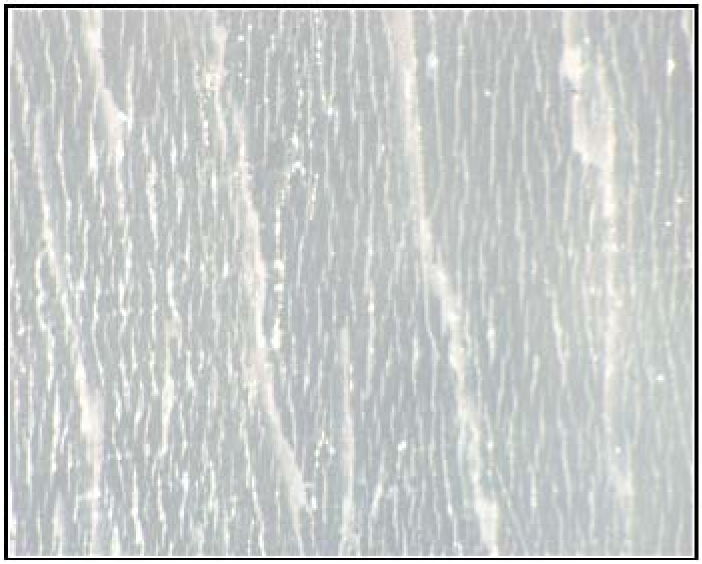


**Figure 5-33: Micro-cracking of UV Exposed Polypropylene (Jones 2002)**

The Building Research Association of New Zealand established four identical exposure sites across the country to determine whether climatic variations, including ultraviolet radiation, have a significant effect on plastics. The plastic samples that were chosen include polyvinyl chloride, low density polyethylene, and polypropylene. The exposure sites were located at Kaitaia, Paraparaumu, the Building Research Association of New Zealand at Judgeford, and Invercargil. According to the results, "There are noticeable trends developing with the tensile strengths of the polyolefins" (Jones 2002). The results of the study are shown in Figure 5-34.

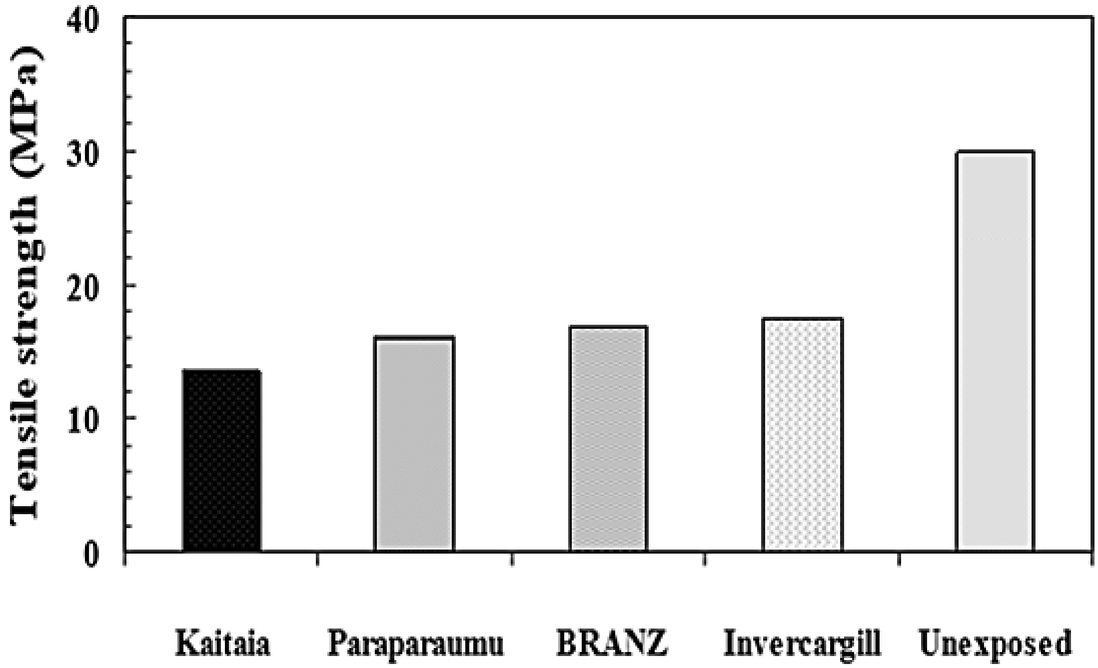



**Figure 5-34: Tensile Strengths of Exposed Polypropylene (Jones 2002)**

Polyvinyl chloride has poor light stability in the wavelength range of 253 to 310 nm and when exposed, the material will develop typical signs of degradation.

> *PVC suffers from poor thermal and light stability. Many factors, such as temperature, humidity and solar radiation cause degradation in polymers. Polyvinyl chloride is a polymer which is very sensitive to the weathering action and this restricts its outdoor applications. This occurs mainly because of changes in mechanical properties and color*. (Hasan and Yousif 2015).

The signs, as listed in "Photostabilization of Poly(Vinyl Chloride) – Still on the Run" (Hasan and Yousif 2015) are as follows:

1. Discoloration from natural to dark-brown or black;
2. Surface cracking;
3. Brittling or softening of the material;

4. Variation of the mechanical properties (tensile strength, ultimate elongation, impact strength, and elasticity modulus);
5. Change of transparency;
6. Formation of a deposit on the material surface.

In an effort to protect against this form of degradation, "UV light absorbers, excited state quenchers, hindered amine light stabilizers, hydro peroxide decomposers, radical scavengers, pigments, fillers and antioxidants" may be applied. However, it is critical the correct ultraviolet stabilizer be selected to suit the exact application. According to CALTRANS's *Highway Design Manual*, "polyvinyl chloride resins used for pipe rarely incorporate ultraviolet protection in amounts adequate to offset long term exposure to direct sunlight." Therefore, the exposed ends of cross drain culverts subject to frequent sunlight exposure can lead to brittleness. The Highway Design Manual recommends avoiding such situations. Figure 5-35 illustrates a section of helical-corrugated metal pipe used as an exposed end piece on polyvinyl chloride pipe. (CALTRANS 2014)

**Figure 5-35: Polyvinyl Chloride Pipe with Metal Pipe Ends (Maher et al. 2015)**

5.6.4 Flammability

All culvert materials are affected by fire and extremely high temperatures. According to the *Buried Facts* article "Fires in Sewers and Culverts" published by the ACPA,

> *Fires in concrete pipe generally do not affect structural strength, flow capacity, or corrosion and abrasion resistance. Metal pipe is usually lined and coated to forestall electrolytic and galvanic corrosion of the pipe wall and to improve hydraulic characteristics. These coatings will burn when exposed to fire. The intense heat can also alter the properties of the metal and result in deflection and loss of structural integrity. Plastic pipe will suffer the same fate as metal, or worse, if the pipe melts and collapses.* (ACPA 1982)

Hancock Concrete Products, a precast concrete manufacturing company in the United States, and the ACPA claim that concrete will not burn. However, extremely high temperatures can affect the compressive strength, flexural strength, and modulus of elasticity of the concrete. The modulus of elasticity is the most sensitive to elevated temperatures of those three. The effects of elevated temperatures are shown in Figure 5-36.

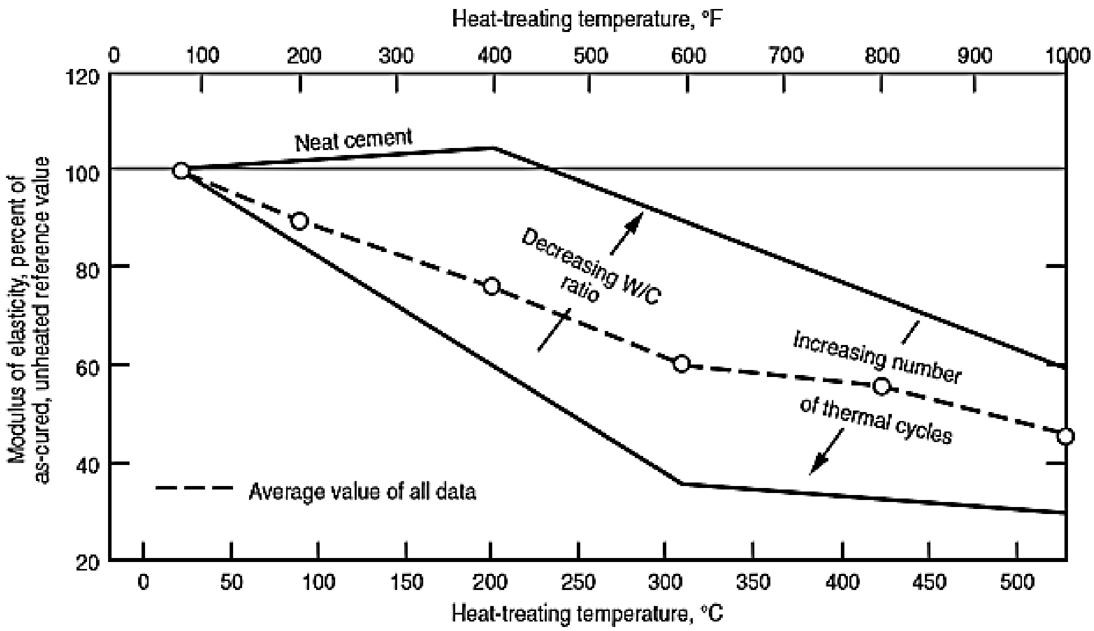



**Figure 5-36: Effects of Elevated Temperatures on Modulus of Elasticity (PCA 2002)**

Mounting concerns regarding the flammability of high density polyethylene prompted the FDOT to conduct a study to determine the actual risk of fire in typical pipe installations. The results of the study indicate that "high-density polyethylene pipe is not at significant risk of fire when installed to present standards and exposed to fire such as that which may be encountered in roadside grass fires." This claim is further supported by *Synthesis 474* which states,

> *In forest fires, all pipe material types can sustain damage from exposure to extremely high temperatures. While thermoplastic pipes would be the most vulnerable, the National Fire Protection Association (NFPA 2012) has given both polyethylene and polypropylene a rating of 1 (Slow Burning) on a scale of 0 to 4, where higher ratings indicate a greater vulnerability*. (Maher et al. 2015)

Despite these assurances, several transportation departments have reported that the pipe ends and the flared end sections of polyethylene pipe have caught on fire as a result of crop, leaf, or controlled burning in roadside ditches. Figure 5-37 illustrates a plastic culvert on fire in Santa Barbara, California. The Michigan Department of Transportation updated their “Culvert and Storm Sewer Pipe Material Policy on Federally Funded Local Agency Projects” in March 2013 to warn of the flammability of polyethylene pipe. As stated in the policy, “In project locations where controlled burning is a common occurrence, concrete or metal culverts may be specified. It may be possible to specify polyethylene culverts as long as a metal or concrete flared end section is also installed” (MDOT 2013).

**Figure 5-37: Plastic Culvert on Fire in Santa Barbara, California (Scully 2015)**

The United States Department of Agriculture examined slip-lining as a possible rehabilitative measure for corrugated metal culverts in the report *Decision Analysis Guide for Corrugated Metal Culvert Rehabilitation and Replacement using Trenchless Technology*. The report listed polyethylene as a possible material for slip-lining, but noted cases of the liner catching on fire. According to *Decision Analysis Guide for Corrugated Metal Culvert Rehabilitation and Replacement using Trenchless Technology*,

> *North Dakota Department of Transportation incurred severe damage to some polyethylene liners installed in corrugated metal pipes due to ditch fires. In 2007, the Cascade Complex fires in the Payette National Forest in Idaho resulted in the destruction of 142 high-density polyethylene culverts ranging in diameter from 18 to 36 inches, 41 wood culvert inlet headwalls, and 50 high-density polyethylene culvert downspouts*. (Matthews et al. 2012)

As a result, the North Dakota Department of Transportation researched alternative liners in the report *Cost Effective Non-Flammable Pipe Liners* (Katti et al. 2003). The fiberglass composites pipes revealed to have the most fire resistance. However, polyethylene liners are more economical. Therefore, the report recommended using high density polyethylene liners with concrete end caps. As a result of the Cascade Complex fires in Idaho, "the Forest Service and the FHWA recommend concrete or masonry headwalls for flammable plastic culverts and liners in forest environments where fire is a possibility" (Matthews et al. 2012). Figure 5-38 depicts one of the burned high density polyethylene culvert inlets at the Cascade Complex in Idaho.

**Figure 5-38: Burned High Density Polyethylene Inlet (Matthews et al. 2012)**

In addition to the lessons learned by Michigan, North Dakota, and Iowa, AASHTO M294 warns that, “When polyethylene pipe is to be used in locations where the ends may be exposed, consideration should be given to protection of the exposed portions due to combustibility of the polyethylene and the deteriorating effects of prolonged exposure to ultraviolet radiation”.

CALTRANS’s *Highway Design Manual* warns of cross drain culverts susceptibility to burning or melting and recommends “either limiting the alternative pipe listing to non-flammable pipe materials or providing a non-flammable end treatment to provide some level of protection.” This is the same warning that most transportation departments issue when allowing high density polyethylene or polypropylene. As stated in the *Highway Design Manual*, *Plastic pipe and pipes with coatings (typically of bituminous or plastic materials) are the most susceptible to damage from fire. Of the plastic pipe types which are allowed, PVC will self-extinguish if the source of the fire is eliminated (i.e., if the grass or brush is consumed or removed) while HDPE can continue to burn as long as an adequate oxygen supply is present*. (CALTRANS 2014)

## 5.7 Hydraulic Design

Before the hydraulic design of a culvert can begin, the design discharge must be estimated. According to CALTRANS's *Highway Design Manual*, "The most important step is to establish the appropriate design storm or flood frequency for the specific site and prevailing conditions" (CALTRANS 2014). The types of flow and control used in the design of highway culverts are: Inlet Control and Outlet Control. Different factors and formulas are used to compute the hydraulic capacity of a culvert based on the type of control. The FHWA's *Hydraulic Design of Highway Culverts* presents the primary design factors associated with each type of control. These design factors are shown in Table 5-41.

**Table 5-41: Factors Influencing Culvert Design (Schall et al. 2012)**

| Factor | Inlet Control | Outlet Control |
|---|---|---|
| Headwater | X | X |
| Area | X | X |
| Shape | X | X |
| Inlet Configuration | X | X |
| Barrel Roughness | – | X |
| Barrel Length | – | X |
| Barrel Slope | X | X |
| Tailwater | – | X |

Note: For inlet control the area and shape factors relate to the inlet area and shape. For outlet control they relate to the barrel area and shape.

Headwater and tailwater refer to specific depths of water measured from the culvert. An example of headwater and tailwater is shown in Figures 5-39 and 5-40, respectively. As stated in Chapter 1 of the FHWA's *Hydraulic Design of Highway Culverts*,

> *The depth of the upstream water surface measured from the invert at the culvert entrance is generally referred to as headwater depth. Tailwater is defined as the depth of water downstream of the culvert measured from the outlet invert. It is an important factor in determining culvert capacity under outlet control conditions*. (Schall et al. 2012)

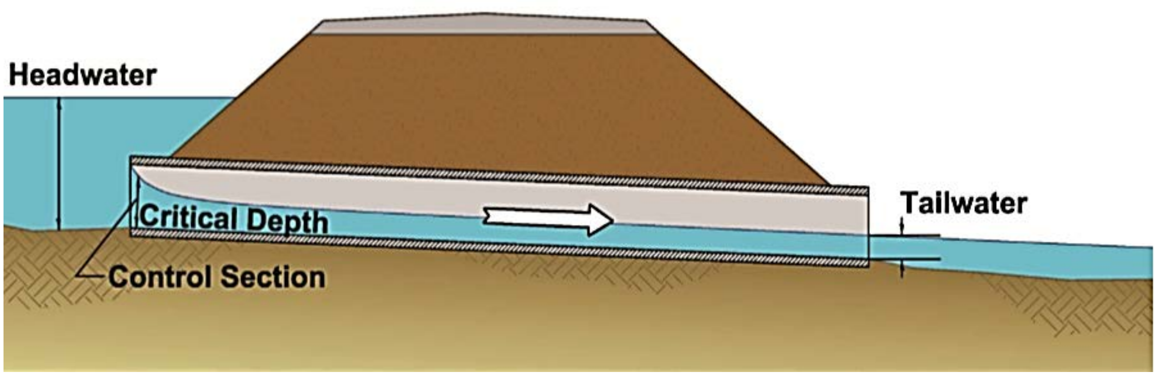



**Figure 5-39: Typical Inlet Flow Control Section (Schall et al. 2012)**

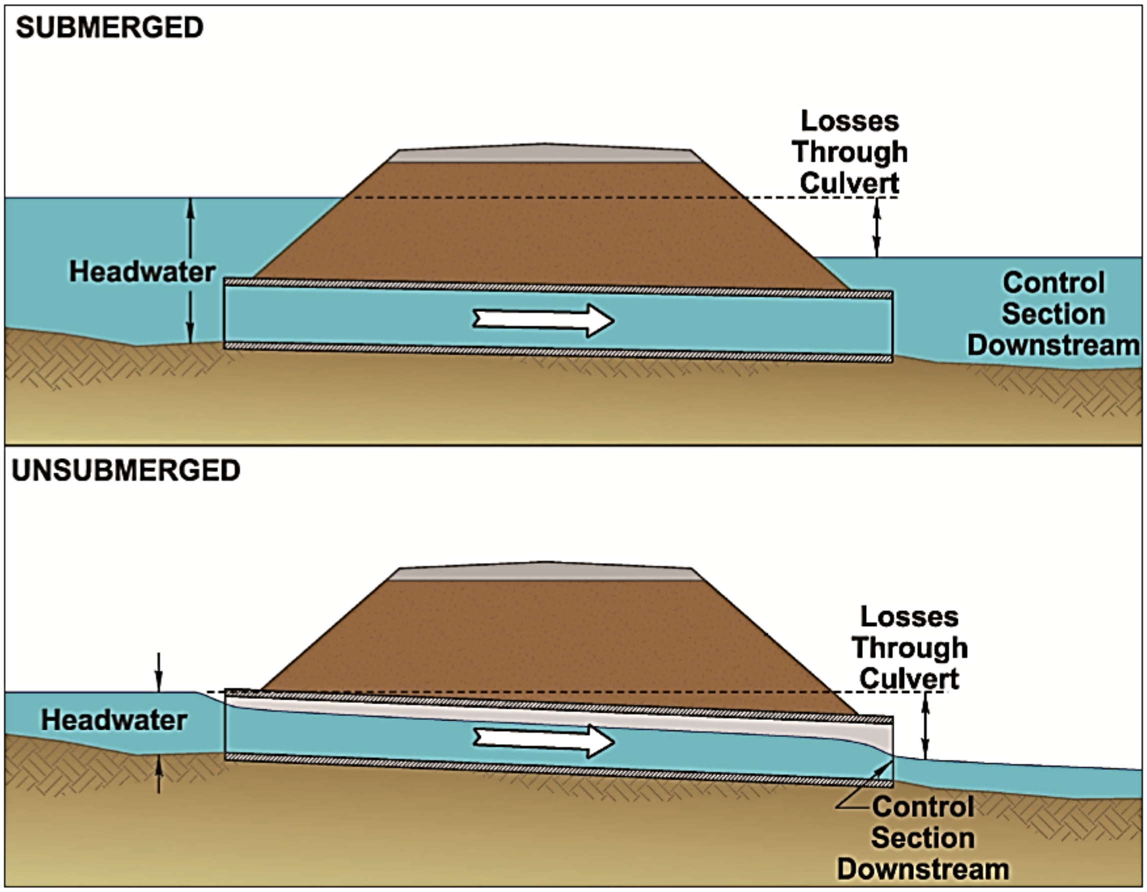



**Figure 5-40: Typical Outlet Control Flow Conditions (Schall et al. 2012)**

5.7.1 Roughness Coefficient

Selecting the correct coefficient of roughness is essential in evaluating the flow and determining the adequate pipe diameter. An excessive coefficient of roughness leads to an uneconomical design and oversizing of the pipe. However, an insufficient coefficient of roughness leads to a hydraulically inadequate pipe. An inadequate hydraulic design leaves culverts susceptible to debris and sediment buildup. This buildup will slowly reduce the capacity of the culvert leading to expensive, time-consuming maintenance and will begin to prohibit the passage of aquatic organisms.

One way to determine the capacity of flow is by the use of Manning's Equation shown below. As stated in ADS's *Drainage Handbook*, "Manning's Equation is the most widely recognized means of determining pipe capacity for gravity flow installations" (ADS 2014). The equation was developed by Irish engineer Robert Manning as an alternative to the Chezy Equation. Manning's Equation assumes uniform flow conditions. While the coefficient of roughness or Manning's n can be calculated, it is often selected from tables.

$$Q = \frac{1.486}{n} \; A \; R^{2/3} \; S^{1/2} \qquad \text{(Equation 5-7)}$$

Where:

$Q$ = pipe capacity
$A$ = cross-sectional flow area
$R$ = hydraulic radius
$S$ = pipe slope
$n$ = coefficient of roughness

The coefficient of roughness has been the focus of extensive research, and recommended values vary by state departments of transportation (Tables 5-42, 5-43, and 5-44). However, the values generally fall within the same range. The most significant variances have been found between laboratory tests and accepted design values. According to the article *Hydraulic Efficiency* (ACPA 2010), laboratory results have been obtained utilizing clean water and straight pipe sections. This leads to wide discrepancies between the coefficient of roughness for smooth wall and rough wall pipes. According to *Manning's n Values History of Research*, *Rough wall, such as unlined corrugated metal pipe have relatively high values which are approximately 2.5 to 3 times those of smooth wall pipe. Smooth wall pipes were found to have values ranging between 0.009 and 0.010 but, historically, engineers familiar with sewers have used 0.012 or 0.013. This "design factor" of 20-30 percent takes into account the difference between laboratory testing and actual installed conditions*. (ACPA 2012)

The coefficient of roughness has been the focus of extensive research, and recommended values vary by state departments of transportation (Tables 5-45, 5-46, and 5-47).

**Table 5-42: Coefficient of Roughness (FDOT 2016)**

| Concrete | Metal | | Plastic (Polyvinyl Chloride, Polyethylene, Polypropylene) | |
|---|---|---|---|---|
| | Helical | Spiral Rib | Single Wall | Double or Triple Wall (smooth interior) |
| 0.012 | 0.020 – 0.024 | 0.012 | 0.024 | 0.012 |

**Table 5-43: Recommended Values for Manning's Coefficient of Roughness**

| American Concrete Pipe Association (ACPA) | National Corrugated Steel Association (NCSA) | Advanced Drainage Systems (ADS) |
|---|---|---|
| **Precast Concrete** | **Corrugated Metal** | **High Density Polyethylene** |
| 0.011 – 0.012 | 0.011 – 0.021 | 0.009 – 0.012 |

**Table 5-44: Manning's *n* Value for Metal Culverts (Schall et al. 2012)**

| Type of Metal Culvert | Roughness or Corrugation | Manning's *n* |
|---|---|---|
| Spiral Rib | Smooth | 0.012 – 0.013 |
| Helical Corrugations | 2-2/3 x ½ inch | 0.011 – 0.023 |
| Helical Corrugations | 6 x 1inch | 0.022 – 0.025 |
| Annular Corrugations | 2-2/3 x ½ inch | 0.022 – 0.027 |

| | | |
|---|---|---|
| Annular Corrugations | 5 x 1 inch | 0.025 – 0.026 |
| Annular Corrugations | 3 x 1 inch | 0.027 – 0.028 |

**Table 5-45: Manning's *n* Values for Culverts (SDDOT 2011)**

| Type of Conduit | Wall Description | Manning's *n* Laboratory | SDDOT Design |
|---|---|---|---|
| Concrete Pipe | Smooth | 0.010-0.011 | 0.012 |
| Concrete Boxes | Smooth | 0.012-0.015 | 0.012 |
| Spiral Rib Metal Pipe | Smooth | 0.012-0.013 | 0.012 |
| Corrugated Metal Pipe, Pipe-Arch and Box | 2 ⅔ x ½ in Annular | 0.022-0.027 | 0.024 |
| | 2 ⅔ x ½ in Helical | 0.011-0.023 | 0.024 |
| | 6 x 1 in Helical | 0.022-0.025 | 0.024 |
| | 5 x 1 in Helical | 0.025-0.026 | 0.024 |
| | 3 x 1 in Helical | 0.027-0.028 | 0.024 |
| | 6 x 2 in Structural Plate | 0.033-0.035 | 0.035 |
| | 9 x 2½ in Structural Plate | 0.033-0.037 | 0.035 |
| Corrugated Polyethylene | Smooth | 0.009-0.015 | 0.012 |
| Corrugated Polyethylene | Corrugated | 0.018-0.025 | 0.024 |
| Polyvinyl Chloride | Smooth | 0.009-0.011 | 0.012 |

**Table 5-46: Manning's *n* Values for Culverts (LaDOTD 2011)**

| Type of Structure | *n* |
|---|---|
| Concrete Pipe | 0.012 |
| Concrete Box | 0.012 |
| Corrugated Metal Pipe | |
| 2⅔ x ½ Corrugations | 0.024 |
| 3 x 1 Corrugations | 0.027 |
| Plastic Pipe | |
| Smooth Flow | 0.009 |
| Corrugated | 0.020 |

**Table 5-47: Manning's *n* Values for Culverts (FDOT 2016)**

| **Type of Structure** | *n* |
|---|---|
| Concrete Box Culverts | 0.012 |
| Concrete Pipes | 0.012 |
| Metal Pipes: | |
| Pipe and Pipe Arch – Helical Fabrication | |
| Re-Corrugated Ends (All Flow Conditions) | |
| 12" to 24" | 0.020 |
| 30" to 54" | 0.022 |
| 60" and larger | 0.024 |
| Pipe and Pipe Arch – Spiral Rib Fabrication | |
| Re-Corrugated Ends (All Flow Conditions) | |
| All Sizes | 0.012 |
| Plastic Pipes: | |
| Polyvinyl Chloride (External Ribs) | |
| All Sizes | 0.012 |
| Polyethylene | |
| Single Wall | 0.024 |
| Double Wall (Smooth) | 0.012 |
| Polypropylene | |
| Single Wall | 0.024 |
| Double and Tripe Wall (Smooth) | 0.012 |

### 5.7.2 Aquatic Organism Passage

Culverts have only recently been designed to consider fish passageways. The desire for hydraulic efficiency often controlled the design. This caused engineers to overlook how the structure might impact the aquatic environment. Figure 5-41 illustrates a culvert design that prevents fish passage while Figure 5-42 illustrates a culvert design that allows fish passage. According to a study performed by North Carolina State University, "Alteration of streams by constructing of road crossing structures can degrade stream habitat leading to a loss of fish spawning sites and an overall reduction of species richness and diversity" (Levine et al. 2007). The report *A Comparison of the Impacts of*

*Culverts versus Bridges on Stream Habitat and Aquatic Fauna* assessed the impacts of culverts and bridges on stream habitat and stream fauna for the North Carolina Department of Transportation.

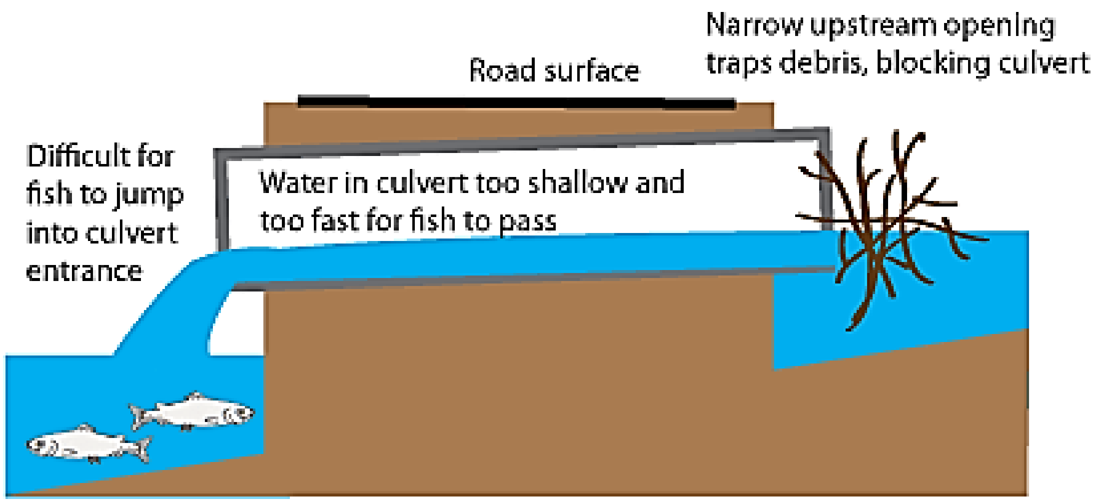



**Figure 5-41: Culvert Design that Prevents Aquatic Organism Passage (NOAA n.d.)**

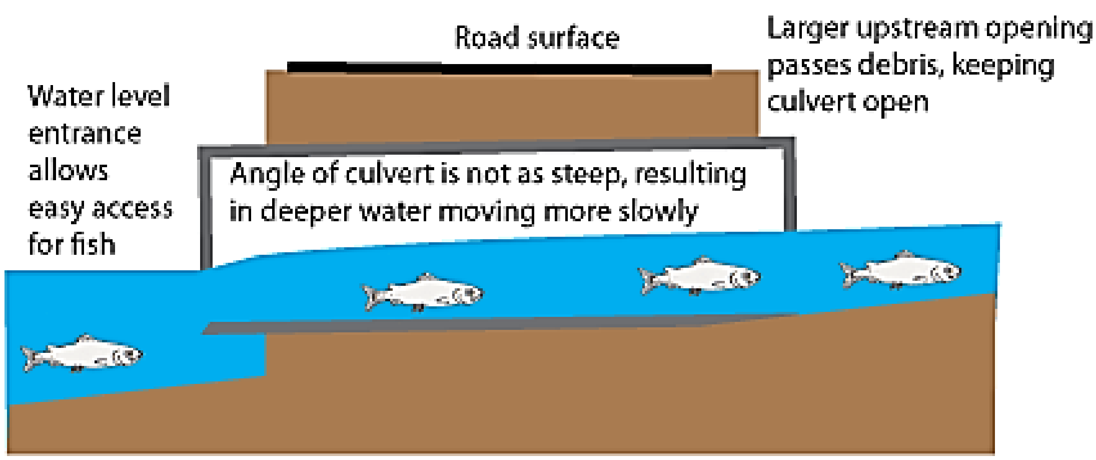



**Figure 5-42: Culvert Design that Allows Aquatic Organism Passage (NOAA n.d.)**

According to *A Comparison of the Impacts of Culverts versus Bridges on Stream Habitat and Aquatic Fauna*, “Culverts are typically the most economically feasible road crossing and potentially the most damaging to biota, stream morphology, and hydraulics”

(Levine et al. 2007). The United States General Accounting Office published the report *Restoring Fish Passage through Culverts and Forest Service and BLM Lands in Oregon and Washington could Take Decades.* The report determined that over half of the 10,000 culverts surveyed on Bureau of Land Management and Forest Service lands in Oregon and Washington are considered barriers to the passage of juvenile salmon.

> *Agency inventory and assessment efforts have already identified nearly 2,600 barrier culverts, but agency officials estimate that more than twice that number may exist. Based on current assessments, the agencies estimate that efforts to restore fish passage may ultimately cost over $375 million and take decades*. (GAO 2001)

The FHWA published two documents to serve as a design aid to facilitate aquatic organism passage in culverts. These documents include: *Design for Fish Passage at Roadway-Stream Crossings: Synthesis Report* and *Culvert Design for Aquatic Organism Passage*. The *Design for Fish Passage at Roadway-Stream Crossings: Synthesis Report* is a compilation of design options, case histories, and assessment techniques to provide an array of appropriate design methods to facilitate fish passage. The document serves only as a reference (Hotchkiss and Frei 2007). The *Culvert Design for Aquatic Organism Passage* presents a stream simulation design procedure as well as methods and best practices for designing culverts to facilitate fish passage (Kilgore et al. 2010). Table 5-48 lists seven types of barriers that have the potential to act as a barrier of fish passage. Figure 5-39 is an example of a metal culvert that prevents fish passage. Figure 5-40 and Figure 5-41 illustrate bottomless culverts designed for aquatic organism passage.

**Table 5-48: Why Culverts are a Barrier of Fish Passage (Hotchkiss and Frei 2007)**

| Barrier Type | Impact |
|---|---|
| Drop | Fish cannot enter structure, can be injured, or will expend too much energy entering the structure to transverse other obstacles |
| Velocity | High velocity exceeds fish swimming causing fish to tire before passing the crossing |
| Turbulence | Fire do not enter culvert or are unable to successfully navigate the waterway |
| Length | Fish may not enter structure due to darkness and fish may fatigue before traversing the structure |
| Depth | Low flow depth causes fish not to be fully submerged causing fish to be unable to swim efficiently or unable to pass the structure |
| Debris | Fish may not be able to pass by debris or constricted flow may create a velocity or turbulence barrier within the culvert |
| Cumulative | Group of culverts, each marginally passable, may be a combined barrier which stresses fish during passage |

**Figure 5-43: Culvert Barrier to Fish Passageway (USDA 2015)**

### 5.8 Minimum and Maximum Diameter

Culverts must be sized appropriately to meet the maximum anticipated site conditions. If a culvert is too small, debris may prevent the flow of water or the passage of aquatic organisms. Other possible failures that could result from incorrect sizing can include flooding, road washouts, blowouts, and erosion. The culvert shown in Figure 5-44 is inadequately sized to pass the large debris moving through the drainage. In addition to rainfall, some culverts must be designed for sudden snowmelt in areas subjected to snow accumulation. The effects of a hurricane may also be considered in the design. These conditions could greatly exceed the pipe's designed capacity if not taken into account.

**Figure 5-44: Inadequately Designed Culvert (Keller 2003)**

According to the Department of the Interior Bureau of Land Management, culverts are typically designed for a minimum 20-year storm event. However, local regulations may require a more conservative design. The double box culvert shown in Figure 5-45 may appear overdesigned. However, it was strategically designed to safely pass the anticipated design flow based upon a hydrological analysis of a 20-year to 50-year storm recurrence event. (Keller 2003)

**Figure 5-45: Reinforced Concrete Box Culvert (Keller 2003)**

In 2004, the WisDOT published the bulletin *Culverts – Proper Use and Installation*. According to the bulletin, "Small increases in diameter can significantly increase culvert capacity. For example, a 30-inch culvert can handle 50% more water than a 24-inch culvert" (WisDOT 2004). The Office of Federal Lands and Highway established the following minimum pipe size criteria to limit maintenance problems due to debris or sedimentation:

- 24-inch or equivalent for cross-road culverts
- 18-inch or equivalent for parallel culverts in roadside ditches and channels

Culvert pipes are available in 6-inch increments. Table 5-49 shows the maximum culvert pipe diameters specified by the ConnDOT.

**Table 5-49: Specified Maximum Pipe Diameters (ConnDOT 2000)**

| Pipe Material | Maximum Diameter |
|---|---|
| Reinforced Concrete | 180 inches |
| Corrugated Steel | 144 inches |
| Corrugated Aluminum | 120 inches |
| Polyethylene | 96 inches |

The following state departments of transportation specify a minimum 18-inch pipe diameter for cross drain applications: Florida, Kentucky, Pennsylvania, Massachusetts, Maine, Georgia, Tennessee, and California. The South Dakota Department of Transportation specifies a 24-inch minimum pipe diameter for cross drain application to avoid construction, maintenance and clogging problems. Table 5-50 shows the minimum size culvert used for cross drain application by the Louisiana Department of Transportation (LaDOTD). Circular and arch cross drain pipes are given the same minimum size. According to LaDOTD's *Hydraulic Manual*,

> *In general, plastic pipes are the same size as concrete pipes, whereas metal pipes are at least one size larger in order to achieve the same hydraulic performance. That is, for diameters up to 60" in diameter, metal pipes will be 6" larger and for diameters 60" and greater, metal pipes will be 12" larger*. (LaDOTD 2011)

**Table 5-50: LaDOTD Minimum Culvert Size (LaDOTD 2011)**

| Location | Structure | Minimum Size |
|---|---|---|
| cross drains | cross drain pipe (arch) | 24-inch diameter or round equivalent arch |
| | reinforced concrete box | 4 x 4 feet |

## 5.9 Minimum and Maximum Soil Cover

"Buried structures shall be designed for force effects resulting from horizontal and vertical earth pressure, pavement load, live load, and vehicular dynamic load allowable" (AASHTO 2008). The amount of load exerted on a culvert is dependent on many factors. According to ConnDOT's *Drainage Manual*,

> *The amount of both dead and live load that is actually exerted on a culvert depends upon whether it is a rigid or flexible material, the height of the embankment above the culvert, the type of material surrounding the culvert, the degree of compaction of the material, and whether special types of structural members are built around the culvert to resist and distribute soil pressures*. (ConnDOT 2000)

Examples of dead loads include the weight of embankment or fill covering the culvert. Examples of live loads include vehicular or pedestrian traffic plus an impact factor. "Wind, temperature, vehicle braking, and centrifugal forces typically have little effect due to earth protection. Structure dead load, pedestrian live load, and ice loads are insignificant in comparison with force effects due to earth fill loading" (AASHTO 2008). The impact factor equation shown below accounts for the rolling motion of the vehicle over a relatively shallow buried pipe. The stationary vehicular load is then multiplied by the impact factor to incorporate additional forces into the design.

$$IM = 33(1.0 - 0.125H) \geq 0\% \qquad \text{(Equation 5-8)}$$

Where:

$IM$ = impact factor, %
$H$ = burial depth, feet

Standard vehicular live loads have been established by AASHTO. These loads are not representative of actual vehicles, but serve as a good approximation based on analysis. The most common vehicular loading for design and analysis include the H and HS standard trucks. Figure 5-46 illustrates typical AASHTO highway loads and Table 5-51 shows the load carried by wheel set. According to CALTRANS' *Bridge Design Specifications*, culverts shall be designed for only HS-20 live loads (CALTRANS 2000). According to the Ohio Department of Transportation's *Bridge Mechanics*, the H and HS standard trucks are defined as follows (ODOT 2014):

- <u>H Loading</u> – H20-44 indicates a 20 ton vehicle with a front axle weighing 4 tons and a rear axle weighing 16 tons. The two axles are spaced 14 feet apart.

- <u>HS Loading</u> – A two unit, three axle vehicle comprised of a highway tractor with a semi-trailer. Spacing from the rear tractor axle can vary from 14 to 30 feet.

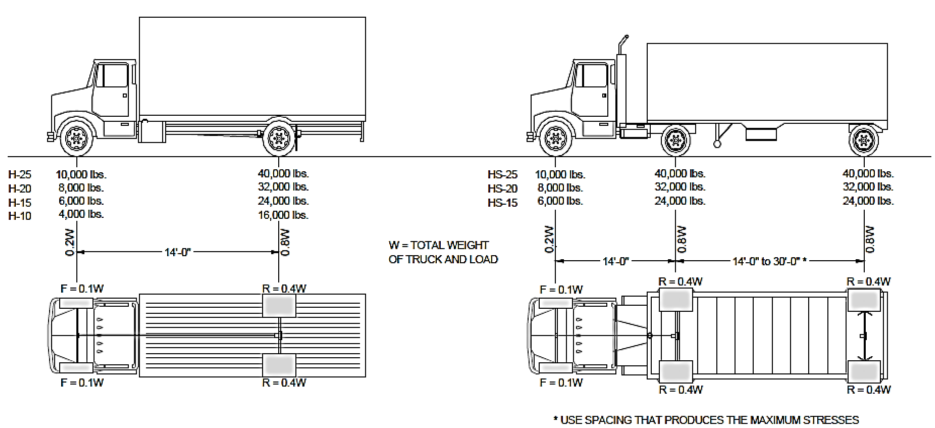



**Figure 5-46: AASHTO Highway Loads (ADS 2015)**

Table 5-51: AASHTO Highway Loads Carried by Wheel Set (ADS 2015)

| Wheel Set | H-10 lbs. (kN) | H-15 or HS-15 lbs. (kN) | H-20 or HS-20 lbs. (kN) | H-25 or HS-25 lbs. (kN) |
|---|---|---|---|---|
| W | 20,000 (89.0) | 30,000 (133.4) | 40,000 (178.0) | 50,000 (222.4) |
| F | 2,000 (8.9) | 3,000 (13.3) | 4,000 (17.8) | 5,000 (22.2) |
| R | 8,000 (35.6) | 12,000 (53.4) | 16,000 (71.2) | 20,000 (89.0) |
| $R_{AXEL}$ | 16,000 (71.1) | 24,000 (106.7) | 32,000 (142.3) | 40,000 (177.9) |

The amount of cover height required is dependent on the pipe material and varies among transportation departments. The Department of the Interior Bureau of Land Management illustrates proper culvert backfill and compaction in Figure 5-47 and recommends the following:

- Metal and plastic culvert pipes have a minimum fill depth of 1 foot
- Concrete culvert pipes have a minimum fill depth of 2 feet

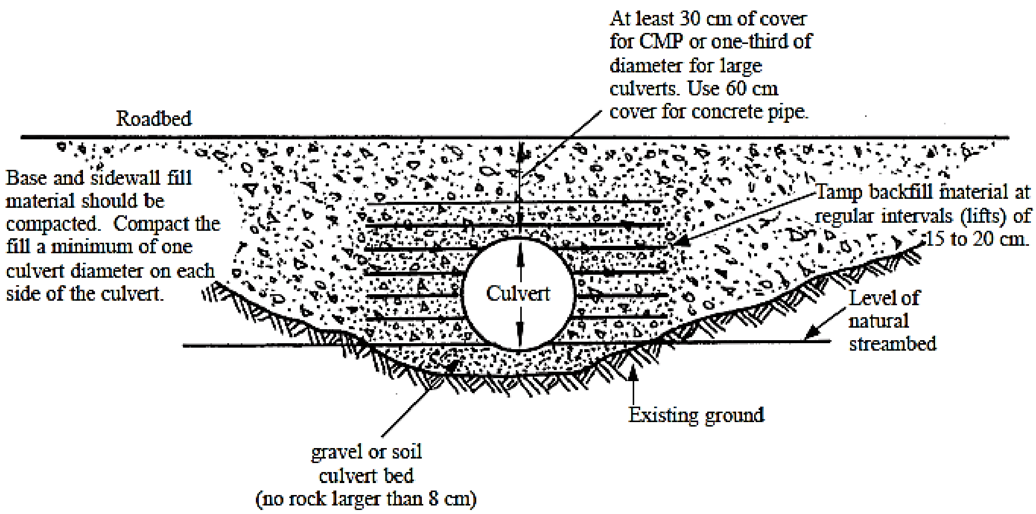



Figure 5-47: Culvert Backfill and Compaction (Keller 2003)

The cover height may be temporarily increased during construction to protect the culvert against heavy equipment. If the weight of the equipment exceeds the design load of the pipe, serious structural problems may occur. “Field tests and analyses prove that the use of heavy construction equipment for compacting or other construction purposes can cause significant stresses and deformations in pipes” (Zhao et al. 1998). According to the report *High Density Polyethylene Pipe Fill Height Table in Arizona* (Ardani et al. 2006), three state transportation departments specified an increase in minimum cover during construction: Alaska, Colorado, and South Carolina. These minimum covers may be found in Table 5-52. Figure 5-48 illustrates the use of a temporary cover to protect the pipe against construction equipment traffic.

**Table 5-52: Minimum Fill Height during Construction for HDPE Pipes (Ardani et al. 2006)**

| State Department of Transportation | Minimum Fill Height during Construction |
|---|---|
| Alaska | 4 feet |
| Colorado | 3 to 4 feet |
| South Carolina | 3 to 4 feet |

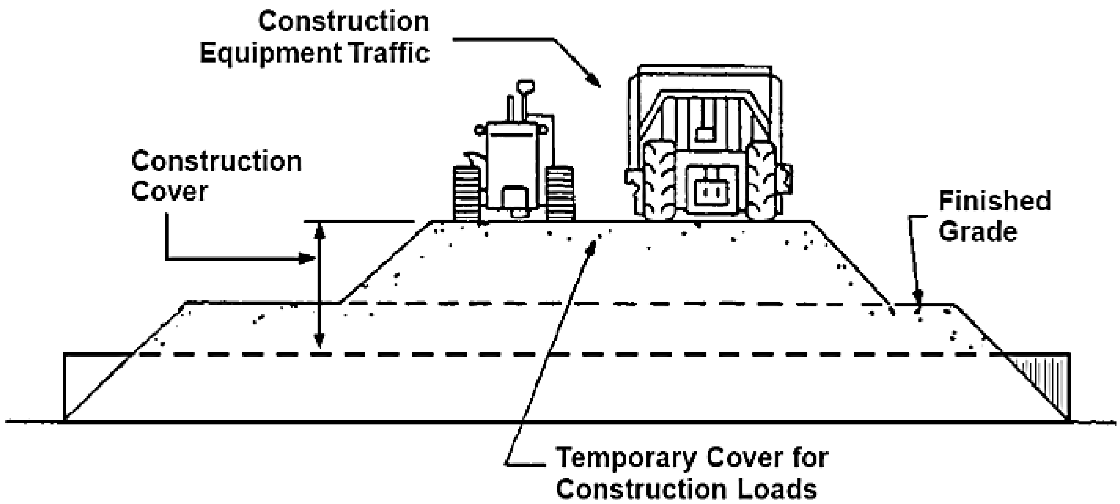



**Figure 5-48: Temporary Cover for Construction Loads (UDOT 2008)**

According to the NYSDOT's *Geotechnical Design Manual*, "Use of extra strong pipe, placement of timbers as bridging to spread the load, or increasing the fill height above the pipe at crossings are precautions which can be taken" (Allen 2013). Appendix C to ConnDOT's *Drainage Manual* requires the engineer to determine the minimum cover for plastic pipe be based on an evaluation of specific site conditions. However, the minimum cover above the pipe shall be at least 3 feet or one pipe diameter in the absence of pipe strength calculations. "The minimum cover should be maintained before allowing vehicles or heavy construction equipment to traverse the pipe trench" (ConnDOT 2000).

The NCDOT's *Drainage User Manual* permits an increase in minimum cover if the Contractor's equipment would cause damage to the completed pipe culvert. As stated in the *Drainage User Manual*,

> *The Specifications require that no heavy equipment shall be allowed to operate over any pipe culvert until the backfill is completed to at least*

*three feet above the top of the pipe. This minimum cover must be maintained until heavy equipment usage is discontinued and the Contractor is prepared to set the final grade*. (NCDOT 2003)

The Corrugated Polyethylene Pipe Association published the guide *Recommended Installation Practices for Corrugated Polyethylene Pipe and Fittings*. According to the guide, the surest solution to protect plastic pipe from unanticipated equipment loads is to reroute construction traffic around the pipe. However, if construction traffic cannot be rerouted, the guide recommends adding at least 3 feet of additional compacted soil over the pipe crown. "This mound can then be graded at the end of construction when heavy traffic is no longer present" (CPPA 2000).

Section 12 of the *AASHTO LRFD Bridge Design Specifications* specifies the minimum soil cover for buried structures and tunnel liners. According to Section 12.6.6.3, "The cover of a well-compacted granular subbase, taken from the top of rigid pavement or the bottom of flexible pavement, shall not be less than that specified in Table 6-5" (AASHTO 2009). AASHTO Table 6-5 is represented by Table 5-53 shown below. These minimum values match the majority of minimum covers specified by the state departments. "Additional cover requirements during construction shall be taken as specified in Article 30.5.5 of the *AASHTO LRFD Bridge Construction Specifications*" (AASHTO 2012).

**Table 5-53: Minimum Soil Cover (AASHTO 2012)**

| Type | Condition | Minimum Cover |
|---|---|---|
| Corrugated Metal Pipe | – | $S/8 \geq 12.0$ inch |
| Spiral Rib Metal Pipe | Steel Conduit | $S/4 \geq 12.0$ inch |
| | Aluminum Conduit | $S/2 \geq 12.0$ inch |
| | Aluminum Conduit | $S/2.75 \geq 24.0$ inch |

| | | |
|---|---|---|
| Structural Plate Pipe Structures | – | $S/8 \geq 12.0$ inch |
| Long-Span Structural Plate Pipe Structures | – | Refer to Table 12.8.3.1.1-1 |
| Structural Plate Box Structures | – | 1.4 feet as specified in Article 12.9.1 |
| Reinforced Concrete Pipe | Unpaved areas and under flexible pavement | $B_c/8$ or $B'_c/8$, whichever is greater, $\geq 12.0$ inch |
| | Compacted granular fill under rigid pavement | 9.0 inch |
| Thermoplastic Pipe | – | $ID/8 \geq 12.0$ inch |

Where,

$S$ = pipe diameter (inch)
$B_c$ = outside diameter or width of the structure (feet)
$B'_c$ = out-to-out vertical rise of pipe (feet)
$ID$ = inside diameter (inch)

5.9.1 Reinforced Concrete

Concrete pipe must meet the requirements of the *AASHTO LRFD Bridge Construction Specifications* or ASTM C1479 *Standard Practice for Installation of Precast Concrete Sewer, Storm Drain, and Culvert Pipe using Standard Installations*. There are four types of Standard Installations for concrete pipe. These types of standard installation are shown in Table 5-54. Each type differs in soil and compaction requirements. According to *LRFD for Fill Height Tables*, "Type 1 bedding provides the most support using highly compacted granular material, while Type 4 provides for less support allowing the use of silts and clay soils with little or no compaction" (ACPA 2013).

**Table 5-54: Installation Soils and Minimum Compaction Requirements (ACPA 2013)**

| Installation Type | Bedding Thickness | Haunch and Outer Bedding | Lower Side |
|---|---|---|---|
| Type 1 | $D_0$/24 minimum, not less than 3 inch. If rock foundation, use $D_0$/12 minimum, not less than 6 inch. | 95% Category I | 90% Category I, 95% Category II, or 100% Category III |
| Type 2 | $D_0$/24 minimum, not less than 3 inch. If rock foundation, use $D_0$/12 minimum, not less than 6 inch. | 90% Category I or 95% Category II | 85% Category I, 90% Category II, or 95% Category III |
| Type 3 | $D_0$/24 minimum, not less than 3 inch. If rock foundation, use $D_0$/12 minimum, not less than 6 inch. | 85% Category I, 90% Category II, or 95% Category III | 85% Category I, 90% Category II, or 95% Category III |
| Type 4 | No bedding required except if rock foundation, use $D_0$/12 minimum, not less than 6 inch. | No compaction required, except if Category III, use 85% | No compaction required, except if Category III, use 85% |

The minimum and maximum fill height varies with each department of transportation. The minimum cover is measured from the top of the pipe to either the bottom of the flexible pavement or to the top of the rigid pavement. As explained in the *Corrugated Steel Pipe Design Manual*,

> *While asphalt does at least as good a job of distributing wheel loads as soil, it is not counted in the minimum cover. The asphalt layer is often very thick and must be placed and compacted in lifts with heavy equipment which would then be on the pipe with inadequate cover. Considering the asphalt thickness as part of the minimum cover could lead to construction problems.* (NCSPA 2008)

The ACPA published *LRFD for Fill Height Tables* using the indirect design method in accordance with Section 12.10.4.3 of the *AASHTO LRFD Bridge Design Specification*, 6th Edition. The fill height tables were based on the following conditions (ACPA 2013):

1. Soil Density = 120 lb/feet$^3$
2. AASHTO HL-93 live load
3. Positive Projecting Embankment Condition. This gives conservative results in comparison to trench conditions.
4. A Type 1 installation requires greater soil stiffness from the surrounding soils than the Type 2, 3, and 4 installations, and is thus harder to achieve.

In recent years, precast concrete manufacturers have encountered contract documents that require AASHTO HL-93 truckloads. However, a comparison between the old HS-20 wheel loads and the new HL-93 wheel loads indicates that the difference is small. According to the National Precast Concrete Association, "The small increases in wheel loads will not affect designs that have excess capacity" (Munkelt 2010).

Table 5-55 illustrates the D-load for Type 1 bedding. The ACPA provides three additional fill height tables for Type 1 Bedding. A maximum fill height of 54 feet is permitted for specially designed concrete pipes. Class V concrete pipes are used for deep installations with a maximum fill height ranging from 31 feet to 53 feet. The burial depth increases as the pipe diameter increases. Therefore, large diameter concrete culverts are used for deep installation.

**Table 5-55: D-Load (lb/feet/feet) for Type 1 Bedding (ACPA 2013)**

| | Fill Height in Feet | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Pipe Size (in) | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 |
| 12 | 1612 | 1399 | 888 | 695 | 633 | 620 | 635 | 661 | 544 | 603 | 662 | 721 | 780 | 839 |
| 15 | 1546 | 1344 | 856 | 673 | 614 | 602 | 617 | 644 | 532 | 589 | 646 | 704 | 761 | 818 |
| 18 | 1462 | 1307 | 836 | 660 | 604 | 593 | 608 | 634 | 526 | 583 | 639 | 696 | 752 | 809 |
| 21 | 1309 | 1281 | 823 | 653 | 598 | 588 | 604 | 630 | 525 | 581 | 637 | 693 | 749 | 805 |
| 24 | 1287 | 1262 | 814 | 648 | 595 | 587 | 603 | 629 | 527 | 583 | 638 | 694 | 750 | 805 |
| 27 | 1230 | 1217 | 789 | 636 | 587 | 582 | 600 | 627 | 530 | 586 | 642 | 697 | 753 | 809 |
| 30 | 1581 | 1272 | 819 | 660 | 605 | 598 | 615 | 640 | 535 | 591 | 646 | 702 | 758 | 814 |
| 33 | 1443 | 1222 | 798 | 651 | 599 | 596 | 615 | 641 | 541 | 597 | 653 | 709 | 765 | 821 |
| 36 | 1329 | 1187 | 780 | 643 | 595 | 595 | 616 | 643 | 547 | 603 | 660 | 716 | 772 | 829 |
| 42 | 1151 | 1099 | 745 | 627 | 587 | 591 | 613 | 641 | 553 | 609 | 665 | 721 | 778 | 834 |
| 48 | 1019 | 961 | 713 | 614 | 582 | 589 | 612 | 641 | 560 | 616 | 673 | 729 | 785 | 841 |
| 54 | 969 | 919 | 689 | 604 | 578 | 589 | 613 | 643 | 569 | 625 | 681 | 737 | 794 | 850 |
| 60 | 994 | 890 | 670 | 596 | 577 | 590 | 615 | 646 | 578 | 634 | 691 | 747 | 804 | 860 |
| 66 | 946 | 865 | 657 | 589 | 576 | 592 | 618 | 651 | 588 | 644 | 701 | 758 | 814 | 871 |
| 72 | 881 | 844 | 647 | 584 | 578 | 595 | 622 | 656 | 598 | 655 | 712 | 769 | 826 | 883 |
| 78 | 827 | 823 | 637 | 582 | 579 | 597 | 625 | 659 | 606 | 663 | 720 | 777 | 834 | 892 |
| 84 | 782 | 805 | 629 | 580 | 580 | 600 | 628 | 664 | 615 | 672 | 729 | 786 | 843 | 901 |
| 90 | 744 | 789 | 622 | 580 | 582 | 603 | 632 | 668 | 712 | 681 | 738 | 795 | 853 | 910 |
| 96 | 712 | 749 | 616 | 580 | 585 | 606 | 637 | 673 | 718 | 690 | 747 | 805 | 862 | 920 |
| 102 | 685 | 723 | 623 | 587 | 592 | 614 | 645 | 682 | 727 | 774 | 757 | 814 | 872 | 929 |
| 108 | 662 | 711 | 629 | 595 | 600 | 623 | 654 | 691 | 736 | 783 | 766 | 824 | 882 | 940 |
| 114 | 642 | 715 | 636 | 603 | 609 | 631 | 663 | 700 | 745 | 793 | 842 | 834 | 892 | 950 |
| 120 | 625 | 720 | 642 | 609 | 617 | 640 | 672 | 709 | 755 | 802 | 852 | 844 | 903 | 961 |
| 126 | 611 | 726 | 649 | 617 | 625 | 649 | 681 | 719 | 764 | 812 | 862 | 913 | 913 | 971 |
| 132 | 599 | 731 | 651 | 625 | 634 | 658 | 690 | 728 | 774 | 822 | 872 | 924 | 976 | 983 |
| 138 | 589 | 736 | 645 | 633 | 643 | 667 | 699 | 738 | 784 | 832 | 883 | 934 | 987 | 994 |
| 144 | 580 | 742 | 651 | 642 | 652 | 676 | 709 | 747 | 794 | 843 | 893 | 945 | 998 | 1052 |

Class I
Class II
Class III
Class IV
Class V
Special Design

Table 5-56 illustrates the D-load for Type 2 bedding. The ACPA provides two additional fill height tables for Type 2 Bedding. A maximum fill height of 42 feet is permitted for specially designed concrete pipes. Class V concrete pipes are used for deep installations with a maximum fill height ranging from 26 feet to 40 feet.

**Table 5-56: D-Load (lb/feet/feet) for Type 2 Bedding (ACPA 2013)**

| Pipe Size (in) | Fill Height in Feet | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 |
| 12 | 1492 | 1322 | 880 | 727 | 694 | 705 | 741 | 788 | 704 | 781 | 858 | 934 | 1011 | 1087 |
| 15 | 1434 | 1272 | 851 | 707 | 676 | 688 | 724 | 771 | 691 | 843 | 841 | 915 | 990 | 1065 |
| 18 | 1358 | 1240 | 834 | 697 | 668 | 680 | 717 | 763 | 688 | 837 | 835 | 909 | 983 | 1056 |
| 21 | 1220 | 1218 | 824 | 692 | 665 | 678 | 715 | 762 | 689 | 839 | 836 | 909 | 983 | 1056 |
| 24 | 1202 | 1203 | 818 | 690 | 665 | 680 | 717 | 764 | 694 | 844 | 841 | 915 | 988 | 1062 |
| 27 | 1151 | 1162 | 796 | 679 | 657 | 675 | 714 | 762 | 696 | 846 | 842 | 915 | 989 | 1062 |
| 30 | 1471 | 1213 | 823 | 701 | 674 | 690 | 727 | 773 | 699 | 850 | 845 | 919 | 992 | 1065 |
| 33 | 1347 | 1168 | 805 | 693 | 669 | 688 | 727 | 773 | 704 | 855 | 850 | 923 | 996 | 1069 |
| 36 | 1244 | 1137 | 789 | 687 | 665 | 687 | 728 | 775 | 710 | 861 | 856 | 929 | 1003 | 1076 |
| 42 | 1084 | 1059 | 759 | 673 | 659 | 685 | 726 | 773 | 715 | 867 | 861 | 933 | 1006 | 1079 |
| 48 | 966 | 935 | 732 | 663 | 655 | 684 | 726 | 774 | 722 | 874 | 867 | 940 | 1013 | 1085 |
| 54 | 923 | 899 | 712 | 655 | 654 | 685 | 728 | 777 | 731 | 884 | 876 | 948 | 1021 | 1094 |
| 60 | 948 | 875 | 696 | 650 | 654 | 688 | 731 | 781 | 740 | 894 | 885 | 958 | 1031 | 1103 |
| 66 | 906 | 855 | 687 | 646 | 655 | 691 | 736 | 787 | 750 | 906 | 896 | 969 | 1041 | 1114 |
| 72 | 850 | 837 | 679 | 643 | 658 | 696 | 741 | 793 | 761 | 918 | 907 | 980 | 1053 | 1126 |
| 78 | 802 | 820 | 672 | 642 | 660 | 697 | 744 | 796 | 768 | 925 | 913 | 986 | 1059 | 1131 |
| 84 | 763 | 805 | 665 | 641 | 661 | 700 | 747 | 799 | 775 | 932 | 920 | 993 | 1065 | 1138 |
| 90 | 730 | 791 | 660 | 641 | 664 | 703 | 750 | 803 | 863 | 940 | 927 | 999 | 1072 | 1144 |
| 96 | 703 | 756 | 655 | 642 | 666 | 706 | 754 | 807 | 867 | 948 | 934 | 1006 | 1078 | 1151 |
| 102 | 679 | 734 | 662 | 649 | 674 | 714 | 761 | 814 | 875 | 1019 | 941 | 1013 | 1086 | 1158 |
| 108 | 660 | 723 | 668 | 657 | 681 | 721 | 769 | 822 | 882 | 1027 | 949 | 1021 | 1093 | 1165 |
| 114 | 643 | 729 | 675 | 665 | 689 | 729 | 776 | 830 | 890 | 1036 | 1016 | 1028 | 1100 | 1172 |
| 120 | 629 | 734 | 682 | 670 | 697 | 737 | 784 | 837 | 898 | 1044 | 1024 | 1036 | 1108 | 1180 |
| 126 | 617 | 740 | 689 | 678 | 705 | 744 | 792 | 845 | 905 | 1053 | 1032 | 1097 | 1115 | 1187 |
| 132 | 607 | 745 | 691 | 686 | 712 | 752 | 800 | 853 | 913 | 1061 | 1039 | 1105 | 1171 | 1195 |
| 138 | 599 | 751 | 686 | 694 | 720 | 760 | 808 | 861 | 921 | 1070 | 1047 | 1112 | 1178 | 1203 |
| 144 | 592 | 757 | 692 | 701 | 728 | 768 | 816 | 869 | 929 | 1079 | 1055 | 1120 | 1186 | 1253 |

Class I
Class II
Class III
Class IV
Class V
Special Design

Table 5-57 illustrates the D-load for Type 3 bedding. The ACPA provides two additional fill height tables for Type 3 bedding. A maximum fill height of 35 feet is permitted for specially designed concrete pipes. Class V concrete pipes are used for deep installations with a maximum fill height ranging from 20 feet to 32 feet.

**Table 5-57: D-Load (lb/feet/feet) for Type 3 Bedding (ACPA 2013)**

| Pipe Size (in) | Fill Height in Feet | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 27 | 28 |
| 12 | 1490 | 1588 | 1686 | 1784 | 1882 | 1980 | 2078 | 2176 | 2274 | 2372 | 2470 | 2568 | 2666 | 2764 |
| 15 | 1450 | 1545 | 1640 | 1735 | 1830 | 1925 | 2020 | 2115 | 2210 | 2305 | 2401 | 2496 | 2591 | 2686 |
| 18 | 1430 | 1523 | 1617 | 1710 | 1803 | 1897 | 1990 | 2083 | 2177 | 2270 | 2363 | 2457 | 2550 | 2643 |
| 21 | 1421 | 1513 | 1606 | 1698 | 1790 | 1883 | 1975 | 2068 | 2160 | 2252 | 2345 | 2437 | 2529 | 2622 |
| 24 | 1419 | 1511 | 1603 | 1695 | 1786 | 1878 | 1970 | 2062 | 2154 | 2246 | 2338 | 2430 | 2521 | 2613 |
| 27 | 1422 | 1514 | 1605 | 1697 | 1789 | 1880 | 1972 | 2064 | 2155 | 2247 | 2339 | 2431 | 2522 | 2614 |
| 30 | 1428 | 1520 | 1612 | 1704 | 1795 | 1887 | 1979 | 2071 | 2162 | 2254 | 2346 | 2437 | 2529 | 2621 |
| 33 | 1437 | 1529 | 1621 | 1713 | 1805 | 1897 | 1989 | 2081 | 2173 | 2265 | 2357 | 2449 | 2541 | 2633 |
| 36 | 1449 | 1541 | 1633 | 1726 | 1818 | 1910 | 2003 | 2095 | 2187 | 2280 | 2372 | 2464 | 2557 | 2649 |
| 42 | 1455 | 1547 | 1639 | 1731 | 1823 | 1915 | 2007 | 2098 | 2190 | 2282 | 2374 | 2466 | 2558 | 2650 |
| 48 | 1465 | 1556 | 1648 | 1740 | 1832 | 1924 | 2016 | 2108 | 2200 | 2291 | 2383 | 2475 | 2567 | 2659 |
| 54 | 1477 | 1569 | 1661 | 1753 | 1845 | 1937 | 2029 | 2121 | 2213 | 2305 | 2397 | 2489 | 2581 | 2673 |
| 60 | 1492 | 1584 | 1676 | 1768 | 1861 | 1953 | 2045 | 2137 | 2229 | 2321 | 2413 | 2506 | 2598 | 2690 |
| 66 | 1508 | 1601 | 1693 | 1786 | 1878 | 1970 | 2063 | 2155 | 2248 | 2340 | 2433 | 2525 | 2617 | 2710 |
| 72 | 1526 | 1619 | 1711 | 1804 | 1897 | 1990 | 2083 | 2175 | 2268 | 2361 | 2454 | 2547 | 2639 | 2732 |
| 78 | 1533 | 1625 | 1718 | 1810 | 1903 | 1995 | 2088 | 2180 | 2273 | 2365 | 2458 | 2550 | 2643 | 2735 |
| 84 | 1540 | 1632 | 1725 | 1817 | 1909 | 2001 | 2094 | 2186 | 2278 | 2370 | 2463 | 2555 | 2647 | 2740 |
| 90 | 1548 | 1640 | 1732 | 1824 | 1916 | 2008 | 2100 | 2192 | 2284 | 2377 | 2469 | 2561 | 2653 | 2745 |
| 96 | 1556 | 1648 | 1740 | 1832 | 1924 | 2016 | 2108 | 2199 | 2291 | 2383 | 2475 | 2567 | 2659 | 2751 |
| 102 | 1565 | 1657 | 1748 | 1840 | 1932 | 2024 | 2115 | 2207 | 2299 | 2390 | 2482 | 2574 | 2666 | 2757 |
| 108 | 1574 | 1666 | 1757 | 1849 | 1940 | 2032 | 2123 | 2215 | 2307 | 2398 | 2490 | 2581 | 2673 | 2764 |
| 114 | 1583 | 1675 | 1766 | 1857 | 1949 | 2040 | 2132 | 2223 | 2315 | 2406 | 2498 | 2589 | 2680 | 2772 |
| 120 | 1593 | 1684 | 1775 | 1866 | 1958 | 2049 | 2140 | 2232 | 2323 | 2414 | 2506 | 2597 | 2688 | 2780 |
| 126 | 1602 | 1693 | 1785 | 1876 | 1967 | 2058 | 2149 | 2241 | 2332 | 2423 | 2514 | 2605 | 2697 | 2788 |
| 132 | 1612 | 1703 | 1794 | 1885 | 1976 | 2067 | 2158 | 2250 | 2341 | 2432 | 2523 | 2614 | 2705 | 2796 |
| 138 | 1622 | 1713 | 1804 | 1895 | 1986 | 2077 | 2168 | 2259 | 2350 | 2441 | 2532 | 2623 | 2714 | 2805 |
| 144 | 1632 | 1722 | 1813 | 1904 | 1995 | 2086 | 2177 | 2268 | 2359 | 2450 | 2541 | 2632 | 2723 | 2814 |

Class I
Class II
Class III
Class IV
Class V
Special Design

Table 5-58 illustrates the D-load for Type 4 bedding. The ACPA provides one additional Fill Height Tables for Type 4 Bedding. A maximum fill height of 25 feet is permitted for specially designed concrete pipes. Class V concrete pipes are used for deep installations with a maximum fill height ranging from 14 feet to 22 feet. Tables 5-59 through 5-65 present the required fill height for concrete pipe specified by various state departments of transportation and the FHWA.

## Table 5-58: D-Load (lb/feet/feet) for Type 4 Bedding (ACPA 2013)

| Pipe Size (in) | Fill Height in Feet | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 |
| 12 | 1579 | 1481 | 1111 | 1032 | 1071 | 1154 | 1264 | 1383 | 1372 | 1521 | 1671 | 1820 | 1969 | 2119 |
| 15 | 1519 | 1426 | 1073 | 998 | 1036 | 1116 | 1221 | 1336 | 1326 | 1616 | 1612 | 1756 | 1899 | 2042 |
| 18 | 1443 | 1391 | 1050 | 978 | 1015 | 1093 | 1195 | 1307 | 1297 | 1580 | 1576 | 1715 | 1854 | 1994 |
| 21 | 1306 | 1366 | 1035 | 966 | 1002 | 1079 | 1179 | 1288 | 1279 | 1557 | 1552 | 1688 | 1825 | 1961 |
| 24 | 1288 | 1349 | 1025 | 959 | 994 | 1070 | 1168 | 1276 | 1267 | 1541 | 1535 | 1670 | 1804 | 1938 |
| 27 | 1238 | 1309 | 1002 | 945 | 982 | 1060 | 1158 | 1265 | 1259 | 1531 | 1524 | 1657 | 1790 | 1922 |
| 30 | 1560 | 1360 | 1029 | 965 | 995 | 1070 | 1166 | 1270 | 1254 | 1524 | 1517 | 1648 | 1780 | 1911 |
| 33 | 1437 | 1316 | 1010 | 955 | 988 | 1064 | 1160 | 1264 | 1252 | 1520 | 1512 | 1642 | 1773 | 1903 |
| 36 | 1336 | 1285 | 993 | 947 | 982 | 1060 | 1157 | 1260 | 1251 | 1518 | 1509 | 1639 | 1768 | 1898 |
| 42 | 1181 | 1211 | 966 | 935 | 976 | 1057 | 1153 | 1256 | 1252 | 1518 | 1508 | 1636 | 1764 | 1892 |
| 48 | 1068 | 1090 | 941 | 927 | 973 | 1056 | 1152 | 1255 | 1257 | 1522 | 1511 | 1638 | 1765 | 1892 |
| 54 | 1029 | 1058 | 925 | 921 | 973 | 1058 | 1154 | 1257 | 1264 | 1529 | 1516 | 1642 | 1768 | 1894 |
| 60 | 1059 | 1038 | 912 | 918 | 975 | 1062 | 1158 | 1261 | 1273 | 1538 | 1523 | 1649 | 1774 | 1899 |
| 66 | 1021 | 1022 | 906 | 917 | 978 | 1066 | 1163 | 1266 | 1282 | 1548 | 1532 | 1657 | 1781 | 1906 |
| 72 | 969 | 1008 | 902 | 917 | 984 | 1072 | 1169 | 1272 | 1292 | 1559 | 1541 | 1666 | 1790 | 1914 |
| 78 | 927 | 996 | 899 | 920 | 990 | 1079 | 1176 | 1280 | 1303 | 1570 | 1551 | 1675 | 1799 | 1923 |
| 84 | 893 | 986 | 898 | 925 | 997 | 1086 | 1184 | 1288 | 1315 | 1582 | 1562 | 1686 | 1810 | 1933 |
| 90 | 866 | 978 | 898 | 931 | 1004 | 1094 | 1192 | 1296 | 1408 | 1595 | 1574 | 1697 | 1820 | 1944 |
| 96 | 844 | 948 | 899 | 936 | 1012 | 1102 | 1201 | 1305 | 1417 | 1608 | 1585 | 1708 | 1831 | 1955 |
| 102 | 826 | 932 | 911 | 949 | 1024 | 1115 | 1214 | 1318 | 1429 | 1685 | 1597 | 1720 | 1843 | 1966 |
| 108 | 812 | 927 | 923 | 962 | 1037 | 1128 | 1226 | 1330 | 1441 | 1698 | 1609 | 1732 | 1855 | 1978 |
| 114 | 801 | 938 | 935 | 975 | 1050 | 1141 | 1239 | 1343 | 1454 | 1712 | 1682 | 1745 | 1867 | 1990 |
| 120 | 793 | 949 | 947 | 986 | 1063 | 1154 | 1252 | 1356 | 1467 | 1726 | 1694 | 1757 | 1879 | 2002 |
| 126 | 786 | 960 | 959 | 999 | 1076 | 1167 | 1265 | 1369 | 1480 | 1740 | 1707 | 1823 | 1892 | 2014 |
| 132 | 782 | 971 | 967 | 1013 | 1090 | 1180 | 1278 | 1382 | 1493 | 1754 | 1720 | 1836 | 1952 | 2027 |
| 138 | 779 | 982 | 968 | 1026 | 1103 | 1194 | 1292 | 1395 | 1506 | 1769 | 1733 | 1848 | 1965 | 2040 |
| 144 | 778 | 994 | 980 | 1039 | 1116 | 1207 | 1305 | 1409 | 1519 | 1783 | 1746 | 1861 | 1978 | 2095 |

Class I
Class II
Class III
Class IV
Class V
Special Design

## Table 5-59: WisDOT's Fill Height for Concrete Pipe (WisDOT 2012)

| Type/ Class of Pipe | AASHTO Materials Designation | Pipe Size I.D. (inches) | Maximum Height of Cover over Top of Pipe (feet) |
|---|---|---|---|
| Reinforced Concrete Class I | M 170 | 60 – 108 | 9 |
| Reinforced Concrete Class II | M 170 | 12 – 108 | 11 |
| Reinforced Concrete Class III | M 170 | 12 – 108 | 15 |
| Reinforced Concrete Class IV | M 170 | 12 – 84 | 25 |
| Reinforced Concrete Class V | M 170 | 12 – 72 | 35 |

**Table 5-60: FDOT's Fill Height for Concrete Pipe (FDOT 2016)**

| Type I Installation | | | | | |
|---|---|---|---|---|---|
| Pipe Diameter (inch) | Maximum Cover (feet) | | | | |
| | Class I | Class II | Class III | Class IV | Class V |
| 12 | 11 | 16 | 22 | 34 | 45 |
| 15 | 12 | 16 | 23 | 34 | 45 |
| 18 | 12 | 16 | 23 | 35 | 45 |
| 24 | 11 | 16 | 22 | 34 | 45 |
| 30 | 11 | 15 | 22 | 34 | 45 |
| 36 | 11 | 15 | 21 | 33 | 45 |
| 42 | 10 | 15 | 21 | 33 | 45 |
| 48 | 10 | 14 | 21 | 32 | 45 |
| 54 | 10 | 14 | 21 | 32 | 45 |
| 60 | 9 | 14 | 20 | 32 | 45 |
| 66 | 9 | 13 | 20 | 31 | 45 |
| 72 | 7 | 12 | 18 | 29 | 45 |
| 78 | 7 | 12 | 18 | 29 | 45 |
| 84 | 7 | 12 | 18 | 29 | 45 |
| 90 | 6 | 11 | 18 | 29 | 45 |
| 96 | 5 | 11 | 18 | 29 | 45 |
| 102 | – | 11 | 17 | 28 | 45 |
| 108 | — | 11 | 17 | 28 | 45 |
| 114 | – | 11 | 17 | 28 | 45 |
| 120 | – | 10 | 17 | 28 | 44 |

Table 5-61: FHWA's Fill Height for Concrete Pipe (AASHTO 2009)

| Pipe Size Diameter Inches | Embankment | | | | | Trench | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Min Cover Inches | Class II | Class III | Class IV | Class V | Class II | Class III | Class IV | Class V |
| | | Maximum Fill Height above Top of Pipe in Feet | | | | | | | |
| 12 | 12 | 11 | 11 | 16 | 23 | 18 | 18 | 26 | 37 |
| 18 | 12 | 10 | 10 | 25 | 39 | 14 | 14 | 31 | 45 |
| 24 | 12 | 11 | 11 | 15 | 31 | 15 | 15 | 22 | 40 |
| 30 | 12 | 9 | 13 | 16 | 35 | 13 | 17 | 20 | 46 |
| 36 | 12 | 9 | 9 | 20 | 41 | 11 | 14 | 26 | 56 |
| 48 | 12 | 12 | 14 | 26 | 44 | 16 | 17 | 31 | 50 |
| 60 | 12 | 15 | 17 | 28 | 44 | 15 | 20 | 32 | 50 |
| 72 | 12 | 13 | 17 | 31 | 41 | 16 | 20 | 35 | 49 |
| 84 | 12 | 13 | 19 | 31 | | 15 | 23 | 37 | |
| 96 | 12 | 13 | 20 | | | 16 | 24 | | |
| 108 | 12 | 16 | 20 | | | 19 | 26 | | |

Table 5-62: WSDOT's Fill Height for Concrete Pipe (WSDOT 2008)

| Pipe Diameter inch | Maximum Cover in Feet | | | | |
|---|---|---|---|---|---|
| | Plain AASHTO M 86 | Class II AASHTO M 170 | Class III AASHTO M 170 | Class IV AASHTO M 170 | Class V AASHTO M 170 |
| 12 | 18 | 10 | 14 | 21 | 26 |
| 18 | 18 | 11 | 14 | 22 | 28 |
| 24 | 16 | 11 | 15 | 22 | 28 |
| 30 | | 11 | 15 | 23 | 29 |
| 36 | | 11 | 15 | 23 | 29 |
| 48 | | 12 | 15 | 23 | 29 |
| 60 | | 12 | 16 | 24 | 30 |

| | | | | | |
|---|---|---|---|---|---|
| 72 | | 12 | 16 | 24 | 30 |
| 84 | | 12 | 16 | 24 | 30 |

Minimum Cover: 2 feet

**Table 5-63: NCDOT's Fill Height for Reinforced Concrete Pipe (NCDOT 2003)**

| Pipe Diameter (in) | Maximum Fill Height (feet) | | |
|---|---|---|---|
| | Class III | Class IV | Class V |
| All Sizes | 23 | 32<br>60* | 90* |

*Use Method "B" Installation under fills greater than 32 feet.

**Table 5-64: SCDOT's Fill Height for Reinforced Concrete Pipe (Gassman 2006)**

| Installation Type[1] | Pipe Diameter (in) | Maximum Height of Fill[2] (feet) | | | Minimum Allowable Cover Height (feet) | |
|---|---|---|---|---|---|---|
| | | Class III | Class IV | Class V | HS-20 Vehicle Loading[3] | Construction Vehicle Loading |
| Type I | 12 – 36 | 27 | 40 | 60 | 1 | 3 |
| | 42 – 66 | 26 | 39 | 58 | 1 | 3 |
| | 72 – 96 | 25 | 38 | 57 | 1 | 3 |
| Type II | 12 – 30 | 19 | 28 | 42 | 1 | 3 |
| | 36 – 96 | 18 | 27 | 41 | 1 | 3 |
| Type III | 12 – 42 | 14 | 22 | 33 | 1 | 3 |
| | 48 – 96 | 13 | 21 | 32 | 1 | 3 |
| Type IV | 12 – 21 | 9 | 14 | 21 | 1 | 3 |
| | 24 – 96 | 9 | 15 | 23 | 1 | 3 |

Notes:
[1]Installation Type is per ASTM C1479 and AASHTO Section 27, Standard Specification for Highway Bridges, Division II: Construction, American Association of State Highway and Transportation Officials, Washington D.C., 2002
[2]Maximum fill heights based on American Concrete Pipe Association (ACPA) Charts
[3]A minimum height of cover is 9 in. is acceptable if pipe is constructed under a rigid pavement and granular backfill is used.

**Table 5-65: ODOT's Fill Height for Reinforced Concrete (ODOT 2015)**

| Pipe Diameter (inches) | Reinforced Concrete Pipe | | | | | |
|---|---|---|---|---|---|---|
| | Class III | | Class IV | | Class V | |
| | Minimum Cover (feet) | Maximum Cover (feet) | Minimum Cover (feet) | Maximum Cover (feet) | Minimum Cover (feet) | Maximum Cover (feet) |
| 12 | 1.5 | 17 | 1.0 | 27 | 0.5 | 41 |
| 15 | 1.5 | 18 | 1.0 | 27 | 0.5 | 42 |
| 18 | 1.5 | 18 | 1.0 | 27 | 0.5 | 42 |
| 21 | 1.5 | 17 | 1.0 | 27 | 0.5 | 42 |
| 24 | 1.5 | 1 | 1.0 | 27 | 0.5 | 42 |
| 27 | 1.5 | 17 | 1.0 | 27 | 0.5 | 41 |
| 30 | 1.5 | 17 | 1.0 | 27 | 0.5 | 41 |
| 33 | 1.5 | 17 | 1.0 | 27 | 0.5 | 41 |
| 36 | 1.5 | 17 | 1.0 | 26 | 0.5 | 41 |
| 42 | 1.5 | 17 | 1.0 | 26 | 0.5 | 41 |
| 48 | 1.5 | 16 | 1.0 | 26 | 0.5 | 41 |
| 54 | 1.5 | 16 | 1.0 | 26 | | |
| 60 | 1.5 | 16 | 1.0 | 26 | | |
| 66 | 1.5 | 16 | 1.0 | 26 | | |
| 72 | 1.5 | 16 | 1.0 | 25 | | |

### 5.9.2 Corrugated Metal

According to the NCSPA's *Corrugated Steel Pipe Design Manual*, "Minimum covers for H20 and H25 highway loads are taken as the greater of span/8 or 12 inches for all corrugated steel pipe except spiral rib pipe" (NCSPA 2008). The NCSPA provides height of cover tables for standard corrugated steel pipe based on the American Iron and

Steel Institute method (Table 5-66). The tables are dependent on pipe shape, wall thickness, and corrugations.

**Table 5-66: NCSPA's Fill Height for Steel Pipe (NCSPA 2008)**

| Height of Cover Limits for Steel Pipe<br>H20 or H25 Live Load · 2-⅔ x ½ Corrugation | | | | | | |
|---|---|---|---|---|---|---|
| Diameter or Span, inch | Min.* Cover, inch | Maximum Cover (feet) for Specified Thickness (inch) | | | | |
| | | 0.064 | 0.079 | 0.109 | 0.138 | 0.168 |
| 12 | 12 | 248 | 310 | | | |
| 15 | 12 | 198 | 248 | | | |
| 18 | 12 | 165 | 206 | | | |
| 21 | 12 | 141 | 177 | 248 | | |
| 24 | 12 | 124 | 155 | 217 | | |
| 30 | 12 | 99 | 124 | 173 | | |
| 36 | 12 | 83 | 103 | 145 | 186 | |
| 42 | 12 | 71 | 88 | 124 | 159 | 195 |
| 48 | 12 | 62 | 77 | 108 | 139 | 171 |
| 54 | 12 | (53) | 67 | 94 | 122 | 150 |
| 60 | 12 | | (57) | 80 | 104 | 128 |
| 66 | 12 | | | 68 | 88 | 109 |
| 72 | 12 | | | (57) | 75 | 93 |
| 78 | 12 | | | (48) | 63 | 79 |
| 84 | 12 | | | (40) | 52 | 66 |
| 90 | 12 | | | (32) | 43 | 54 |
| 96 | 12 | | | | 35 | 45 |

Notes:

1. Fill heights in parentheses require standard trench installation; all others may be embankment or trench.
2. 12 inch through 36 inch diameter, heavier gages may be available.

* Minimum covers are measured from top of pipe to bottom of flexible pavement or top of pipe to top of rigid pavement. Minimum covers must be maintained in unpaved traffic areas.

Table 5-67 through 5-72 present the required fill heights for corrugated steel pipe and corrugated aluminum pipe specified by various departments of transportation. State departments of transportation include Maine, Florida, Washington, Oregon, and New York.

**Table 5-67: MaineDOT's Fill Height for Corrugated Steel (MaineDOT 2005)**

| Round Pipe – 2-⅔ x ½ Corrugations | | | | |
|---|---|---|---|---|
| **Pipe Diameter (in)** | **Standard Thick (in)/ Height of Fill (feet)** | **Non-Standard Thick./Height of Fill** | **Non-Standard Thick./Height of Fill** | **Non-Standard Thick./Height of Fill** |
| 12 and 15 | 0.064/1.5 – 45 | | | |
| 18 | 0.064/1.5 – 35 | 0.079/35 – 55 | | |
| 21 | 0.064/1.5 – 35 | 0.079/35 – 50 | 0.109/50 – 55 | |
| 24 | 0.064/1.5 – 20 | 0.079/20 – 40 | 0.109/40 – 50 | 0.138/50 – 60 |
| 30 | 0.079/1.5 – 25 | 0.109/25 – 40 | 0.138/25 – 45 | 0.168/55 – 60 |
| 36 | 0.079/1.5 – 15 | 0.109/15 – 25 | 0.138/25 – 45 | 0.168/45 – 60 |
| 42 | 0.109/1.5 – 20 | 0.138/20 – 35 | 0.168/35 – 60 | |
| 48 | 0.109/1.5 – 25 | 0.138/20 – 50 | 0.168/50 – 60 | |
| 54 | 0.109/1.5 – 20 | 0.138/20 – 40 | 0.168/40 – 50 | |
| 60 | 0.138/1.5 – 25 | 0.168/25 – 45 | | |
| 66 | 0.138/1.5 – 20 | 0.168/20 – 40 | | |
| 72 | 0.168/1.5 – 30 | | | |

Table 5-68: FDOT's Fill Height for Corrugated Aluminum (FDOT 2016)

| Round Pipe – 2-⅔ x ½ Corrugations | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| D (inch) | Area (sq. feet) | Minimum Cover (inch) | | | | | Maximum Cover (feet) | | | | |
| | | Sheet Thickness in Inches (Gage) | | | | | Sheet Thickness in Inches (Gage) | | | | |
| | | 0.06 | 0.075 | 0.105 | 0.135 | 0.164 | 0.06 | 0.075 | 0.105 | 0.135 | 0.164 |
| | | (16) | (14) | (12) | (10) | (8) | (16) | (14) | (12) | (10) | (8) |
| 12 | 0.8 | 12 | 12 | NA | NA | NA | 100+ | 100+ | NA | NA | NA |
| 15 | 1.2 | 12 | 12 | NA | NA | NA | 100+ | 100+ | NA | NA | NA |
| 18 | 1.8 | 12 | 12 | 12 | NA | NA | 83 | 100+ | 100+ | NA | NA |
| 21 | 2.4 | 12 | 12 | 12 | NA | NA | 71 | 89 | 100+ | NA | NA |
| 24 | 3.1 | 12 | 12 | 12 | NA | NA | 62 | 78 | 100+ | NA | NA |
| 30 | 4.9 | 12 | 12 | 12 | NA | NA | 50 | 62 | 87 | NA | NA |
| 36 | 7.1 | NS | 12 | 12 | 12 | NA | NS | 52 | 73 | 94 | NA |
| 42 | 9.6 | NS | NS | 12 | 12 | NA | NS | NS | 62 | 80 | NA |
| 48 | 12.6 | NS | NS | 12 | 12 | 12 | NS | NS | 54 | 70 | 86 |
| 54 | 15.9 | NS | NS | NS | 12 | 12 | NS | NS | NS | 62 | 76 |
| 60 | 19.6 | NS | NS | NS | NS | NS | NS | NS | NS | NS | NS |
| 66 | 23.8 | NS | NS | NS | NS | NS | NS | NS | NS | NS | NS |
| 72 | 28.3 | NS | NS | NS | NS | NS | NS | NS | NS | NS | NS |

NA – Not Available
NS – Not Suitable (For Highway LRFD HL-93 Live Loadings)

Table 5-69: WSDOT's Fill Height for Corrugated Steel (WSDOT 2008)

| Round Pipe – 2-⅔ x ½ Corrugations | | | | | |
|---|---|---|---|---|---|
| Pipe Diameter (inch) | Maximum Cover in Feet | | | | |
| | 0.064 inch | 0.079 inch | 0.109 inch | 0.138 inch | 0.168 inch |
| 12 | 100 | 100 | 100 | 100 | |
| 18 | 100 | 100 | 100 | 100 | |
| 24 | 98 | 100 | 100 | 100 | 100 |

| | | | | | |
|---|---|---|---|---|---|
| 30 | 78 | 98 | 100 | 100 | 100 |
| 36* | 65 | 81 | 100 | 100 | 100 |
| 42* | 56 | 70 | 98 | 100 | 100 |
| 48* | 49 | 61 | 86 | 100 | 100 |
| 54* | | 54 | 76 | 98 | 100 |
| 60* | | | 68 | 88 | 100 |
| 66* | | | | 80 | 98 |
| 72* | | | | 73 | 90 |
| 78* | | | | | 80 |
| 84* | | | | | 69 |

* Designers should consider the most efficient corrugation for the pipe diameter.
Minimum Cover: 2 feet

**Table 5-70: WSDOT's Fill Height for Corrugated Aluminum (WSDOT 2008)**

| Round Pipe – 2-⅔ x ½ Corrugations | | | | | |
|---|---|---|---|---|---|
| Pipe Diameter (inch) | Maximum Cover in Feet | | | | |
| | 0.060 inch | 0.075 inch | 0.105 inch | 0.135 inch | 0.164 inch |
| 12 | 100 | 100 | | | |
| 18 | 75 | 94 | 100 | | |
| 24 | 56 | 71 | 99 | | |
| 30 | | 56 | 79 | | |
| 36 | | 47 | 66 | 85 | |
| 42 | | | 56 | 73 | |
| 48 | | | 49 | 63 | 78 |
| 54 | | | 43 | 56 | 69 |
| 60 | | | | 50 | 62 |
| 66 | | | | | 56 |
| 72 | | | | | 45 |

Minimum Cover: 2 feet

**Table 5-71: ODOT's Fill Height for Aluminum (ODOT 2015)**

| Helical · 2-⅔ x ½ | | | | | | | |
|---|---|---|---|---|---|---|---|
| Pipe Diameter (inch) | Minimum Cover (feet) | Lock Seam | | | | | Minimum Cover (feet) |
| | | Specified Thickness (inch) | | | | | |
| | | 0.060 | 0.075 | 0.105 | 0.135 | 0.164 | |
| | | Maximum Cover (feet) | | | | | |
| 12 | 1.0 | 100 | 100 | 100 | | | 1.0 |
| 15 | 1.0 | 100 | 100 | 100 | | | 1.0 |
| 18 | 1.0 | 84 | 100 | 100 | | | 1.0 |
| 21 | 1.0 | 72 | 90 | 100 | | | 1.0 |
| 24 | 1.0 | 63 | 78 | 100 | 100 | 100 | 1.0 |
| 30 | 1.0 | | 63 | 88 | 100 | 100 | 1.0 |
| 36 | 1.0 | | 52 | 73 | 94 | 100 | 1.0 |
| 42 | 1.5 | | | 63 | 81 | 99 | 1.0 |
| 48 | 1.5 | | | 55 | 71 | 86 | 1.0 |
| 54 | 1.5 | | | 48 | 63 | 77 | 1.0 |
| 60 | 1.5 | | | | 52 | 65 | 1.0 |
| 66 | 1.5 | | | | | 53 | 1.5 |
| 72 | 1.5 | | | | | 43 | 1.5 |

**Table 5-72: NYSDOT's Fill Height for Aluminum (NYSDOT 2014)**

| Corrugated Aluminum Pipe[1,2] | | | | | |
|---|---|---|---|---|---|
| Pipe Diameter (inch) | Minimum Fill Height to Subgrade Surface (feet) | Maximum Allowable Height of Cover[3] (feet) | | | |
| | | Gauge for 2-½ x ½ Corrugation | | | |
| | | 16 | 14 | 12 | 10 |
| 12 | 1 | 50 | 50 | 86 | Not |

| | | | | | |
|---|---|---|---|---|---|
| 15 | | 40 | 40 | 69 | Not Recommended[4] |
| 18 | | 33 | 33 | 57 | |
| 21 | | 28 | 28 | 49 | |
| 24 | | 25 | 25 | 43 | |
| 27 | | 22 | 22 | 38 | 40 |
| 30 | | – | 20 | 34 | 36 |

Notes:
[1] Gauge, diameter, and corrugation combinations not included in this table shall not be specified.
[2] HS-25 Live Loading.
[3] Maximum vertical distance between the top of the pipe and finished or surcharge grade.
[4] Gauge, diameter, and corrugation combinations do not meet structural criteria, or are not manufactured.

### 5.9.3 Plastic

Thermoplastic pipe must meet the requirements of the *AASHTO LRFD Bridge Construction Specifications* or ASTM D2321 *Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications*. According to the guideline “Design Methodology” published by the PPI, “Pipe in traffic areas should have at least 1 foot of cover over the pipe crown for 4-inch to 48-inch diameter pipe and 1.5 feet of cover for 54-inch and 60-inch diameter pipe” (PPI n.d.). Table 5-73 presents the maximum fill heights for high density polyethylene pipe specified by the PPI.

**Table 5-73: Maximum Cover Heights for HDPE Pipe (feet) (PPI n.d.)**

| | **Class I** | | **Class II** | | | | **Class III** | | |
|---|---|---|---|---|---|---|---|---|---|
| **Pipe Dia. (inch)** | **Uncompacted** | **Compacted** | **85%** | **90%** | **95%** | **100%** | **85%** | **90%** | **95%** |
| 4 | 17 | 59 | 17 | 24 | 37 | 59 | 15 | 18 | 24 |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 6 | 16 | 57 | 16 | 24 | 36 | 57 | 15 | 17 | 24 |
| 8 | 14 | 51 | 14 | 21 | 32 | 51 | 13 | 15 | 22 |
| 10 | 13 | 50 | 13 | 20 | 31 | 50 | 12 | 14 | 21 |
| 12 | 13 | 49 | 13 | 20 | 31 | 49 | 12 | 14 | 21 |
| 15 | 13 | 49 | 13 | 20 | 31 | 49 | 12 | 14 | 21 |
| 18 | 13 | 49 | 13 | 20 | 31 | 49 | 12 | 14 | 21 |
| 24 | 13 | 51 | 13 | 21 | 32 | 51 | 12 | 14 | 21 |
| 30 | 13 | 51 | 13 | 21 | 32 | 51 | 12 | 14 | 21 |
| 36 | 13 | 50 | 13 | 20 | 31 | 50 | 12 | 14 | 21 |
| 42 | 11 | 47 | 11 | 19 | 29 | 47 | 10 | 13 | 19 |
| 48 | 11 | 46 | 11 | 18 | 29 | 46 | 10 | 12 | 19 |
| 54 | 11 | 44 | 11 | 18 | 28 | 44 | 10 | 12 | 18 |
| 60 | 11 | 45 | 11 | 18 | 28 | 45 | 10 | 12 | 19 |

Note: Alternate backfill materials and compaction levels not shown in the table may also be acceptable. This is a general guideline. Contact the manufacturer for further detail.
*All cover heights measured in feet.

The North Carolina Department of Transportation requires a minimum cover of 12 inches and maximum cover of 20 feet for plastic pipe ranging from 12 inches to 48 inches in diameter (NCDOT 2003). The Wisconsin Department of Transportation requires a minimum cover of 24 inches and maximum cover of 11 feet for corrugated polyethylene plastic pipe and 15 feet for corrugated polypropylene plastic pipe ranging from 12 inches to 36 inches in diameter (WisDOT 2012). The New York Department of Transportation requires a minimum cover to subgrade surface of 12 inches and maximum cover of 15 feet for corrugated polyethylene plastic pipe ranging from 12 inches to 60 inches in diameter based upon HS-25 live loading (NYSDOT 2014). Tables 5-74 through 5-78 present additional required fill height for plastic pipe specified by various state departments of transportation.

**Table 5-74: WSDOT's Fill Height Table for Plastic (WSDOT 2008)**

| Solid Wall PVC | Profile Wall PVC | Corrugated Polyethylene |
|---|---|---|
| ASTM D 3034 SDR 35<br>3 inch to 15 inch dia.<br><br>ASTM F 679 Type 1<br>18 inch to 48 inch dia. | AASHTO M 304<br>Or<br>ASTM F 794 Series 46<br>4 inch to 48 inch dia. | AASHTO M 294 Type S<br>12 inch to 60 inch dia. |
| 25 feet<br>All diameters | 25 feet<br>All diameters | 25 feet<br>All diameters |

Note: Minimum Cover: 2 feet

**Table 5-75: FDOT's Minimum Cover for Corrugated Plastic Pipe (FDOT 2016)**

| Pipe Material | Pipe Diameter (inch) | Minimum Cover (inch) |
|---|---|---|
| Corrugated Polyethylene and Corrugated Polypropylene | 12 to 48 | 24 |
| | 60 | 30 |

**Table 5-76: FDOT's Maximum Cover for Corrugated Polyethylene (FDOT 2016)**

| Diameter (inch) | Maximum Cover (feet) |
|---|---|
| 12 | 19 |
| 15 | 20 |
| 18 | 17 |
| 24 | 13 |
| 30 | 13 |
| 36 | 14 |

| | |
|---|---|
| 42 | 13 |
| 48 | 12 |
| 60 | 13 |

**Table 5-77: FDOT's Maximum Cover for Corrugated Polypropylene (FDOT 2016)**

| Diameter (inch) | Maximum Cover (feet) |
|---|---|
| 12 | 21 |
| 15 | 22 |
| 18 | 19 |
| 24 | 16 |
| 30 | 19 |
| 36 | 16 |
| 42 | 15 |
| 48 | 15 |
| 60 | 16 |

**Table 5-78: NYSDOT's Fill Height for Polypropylene (NYSDOT 2014)**

| Polypropylene[1,3] | | |
|---|---|---|
| Diameter (inch) | Minimum Fill Height to Subgrade Surface (feet) | Maximum Allowable Height of Cover (feet[2]) |
| 12 | 1 | 24 |
| 15 | 1 | 28 |
| 18 | 1 | 21 |
| 24 | 1 | 18 |
| 30 | 1 | 19 |
| 36 | 1 | 18 |

| | | |
|---|---|---|
| 42 | 1 | 20 |
| 48 | 1 | 18 |
| 60 | 2 | 20 |

Notes:
[1] HL-93 live loading.
[2] Maximum vertical distance between the top of the pipe and the finished or surcharge grade.
[3] Applicable to Type S and Type D pipe.

In 2006, the Arizona Department of Transportation (ADOT) sponsored a nationwide survey to determine the recommended use of high density polyethylene pipe for roadway application. The survey requested pipe diameter, minimum and maximum fill heights, and backfill material. According to the report *High Density Polyethylene Pipe Fill Height* (Ardani et al. 2006),

1. The most prevalent sizes of pipes ranged from 12 inches to 48 inches;
2. 54-inch to 60-inch diameters have only recently been approved by AASHTO.

All 50 state departments of transportation were asked to participate in the survey. Forty-seven state departments of transportation responded. Table 5-79 shows the minimum and maximum pipe diameter allowed by each state department of transportation. The maximum fill heights varied between departments. Fill heights ranged from a few feet to over 50 feet. The most commonly used fill heights reported were 10 feet and 20 feet. Of the 47 departments of transportation that responded, 11 had no specified maximum fill height. The maximum fill heights specified by each state department of transportation are shown in Figure 5-49.

Table 5-79: ADOT Survey Data (Ardani et al. 2006)

| State | HDPE Pipe Diameter | | State | HDPE Pipe Diameter | |
|---|---|---|---|---|---|
| | Minimum | Maximum | | Minimum | Maximum |
| | Inch | Inch | | Inch | Inch |
| Alabama | 12 | 48 | Missouri | 12 | 60 |
| Alaska | 12 | 48 | Montana | Only Allow 18 | |
| Arkansas | No Standard Criteria | | Nebraska | 12 | 36 |
| California | | 60 | Nevada | No Standard Criteria | |
| Colorado | 12 | 48 | New Jersey | No Standard Criteria | |
| Connecticut | 12 | 48 | New Mexico | 12 | 60 |
| Delaware | 8 | 48 | New York | 12 | 48 |
| District of Columbia | HDPE Pipes Not Used | | North Carolina | 12 | 48 |
| Florida | 15 | 48 | Ohio | 12 | 60 |
| Georgia | 12 | 36 | Oklahoma | 18 | 60 |
| Hawaii | 18 | 60 | Oregon | 12 | 60 |
| Idaho | 12 | 48 | Rhode Island | 12 | 48 |
| Illinois | | 36 | South Carolina | 12 | 60 |
| Indiana | 12 | 36 | South Dakota | 18 | |
| Iowa | 24 | 48 | Tennessee | 12 | 48 |
| Kansas | No Standard Criteria | | Texas | 18 | 48 |
| Kentucky | 12 | 48 | Utah | 18 | 60 |
| Louisiana | 12 | 48 | Vermont | 12 | 48 |
| Maine | 12 | 48 | Virginia | 12 | 48 |
| Massachusetts | 6 | 36 | Washington | 12 | 60 |
| Michigan | 12 | 36 | West Virginia | 12 | 48 |
| Minnesota | 12 | 36 | Wisconsin | | 36 |
| Mississippi | 12 | 36 | Wyoming | No Standard Criteria | |

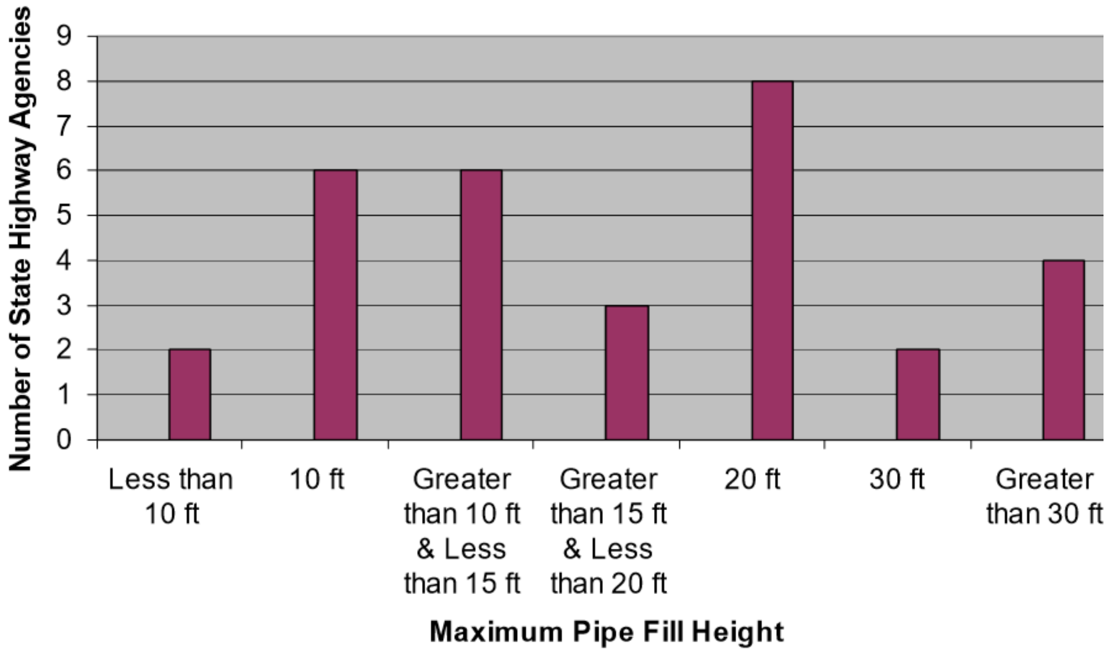



**Figure 5-49: Surveyed Maximum Fill Heights (Ardani et al. 2006)**

In May 1988, David B. Woodham published the report *Plastic Pipe Use Under Highways*. The report evaluated the installation and performance of PVC storm sewer pipes installed beneath major highways in Alamosa, Colorado. The pipe diameters ranged from 8 to 24 inches, and the fill height was limited to 15 feet. The pipes were observed for approximately three and a half years. Table 5-80 shows the total quantities of polyvinyl chloride storm sewer pipe installed.

**Table 5-80: Total Quantities of Polyvinyl Chloride Sewer Pipe (Woodham 1988)**

| Pipe Diameter (inch) | Quantity (linear feet) |
|---|---|
| 8 | 195 |
| 12 | 88 |
| 15 | 2780 |
| 18 | 1455 |
| 21 | 480 |
| 24 | 1552 |

The PVC pipes were installed in accordance with the same installation procedures used for other types of pipe. Most of the sewer pipes were placed in the trench by hand. “The project engineer reported that the plastic pipes are more prone to damage than concrete or steel pipes but that given reasonable care the plastic pipes are sufficiently durable and in addition, are much easier to handle” (Woodham 1988). The only difficulty observed by the contractor was during backfill operations. According to the report, the pipes tended to “float” as the backfill material was placed and tamped. This floating was believed to have been caused by the light weight of the plastic pipes.

No damage or deformation of the pipes was observed during any of the scheduled inspections. The pipe ends were reported to be in “good condition with no chips or visible cracks.” A cost analysis was performed and revealed a substantial savings of $52,000. Table 5-81 depicts the price per linear foot of polyvinyl chloride pipe, reinforced concrete pipe, and corrugated steel pipe during the time of installation. In addition to reduced material cost, cost savings were also a result of handling ease. While some additional costs were reported due to the contractor’s unfamiliarity of polyvinyl chloride, the material savings were substantial enough to justify further consideration of plastic pipe.

**Table 5-81: Comparison of Costs per Linear Foot of Pipe (Woodham 1988)**

| Pipe Diameter | Polyvinyl Chloride* | Reinforced Concrete** | Corrugated Steel** |
|---|---|---|---|
| 8 inch | $5.63 | N/A | N/A |
| 12 inch | $8.70 | $11.50 | $17.00 |
| 15 inch | $11.90 | $22.40 | $22.54 |
| 18 inch | $16.52 | $22.74 | $18.00 |
| 21 inch | $14.50 | $24.14 | N/A |
| 24 inch | $24.65 | $30.30 | $36.00 |

N/A Particular pipe was not included in any projects in 1984/1985
* Weighted average of pipes installed in Alamosa projects
** Weighted average from 1984 and 1985 Colorado Cost Data Manuals

Woodham concluded that polyvinyl chloride pipes should be considered in highway construction, and plastic pipes may prove cost effective when installed in corrosive environments. "At this time there appears to be no reason for disallowing the use of plastic pipes in highway construction. The plastic pipes will continue to be an experimental feature until sufficient long-term data has been acquired" (Woodham 1988). However, several questions were posed concerning the long-term performance of plastic pipe. These questions have been reprinted below.

1. Do plastic pipes creep? If so, under what fill heights does this become a problem? What is the rate of creep? Will the larger plastic pipes now being manufactured (> 24") creep more?

2. Are there construction techniques which can eliminate "floating" during backfill operations?

3. Are connections between pipes as durable as the pipes?

4. Is ultraviolet degradation (on end sections) a long-term problem?

Figure 5-50 illustrates the completed backfill operation for a 21-inch polyvinyl chloride sewer pipe. Figure 5-51 shows a chipped 21-inch polyvinyl chloride pipe.


**Figure 5-50: 21-inch Polyvinyl Chloride Sewer Pipe after Backfill (Woodham 1988)**

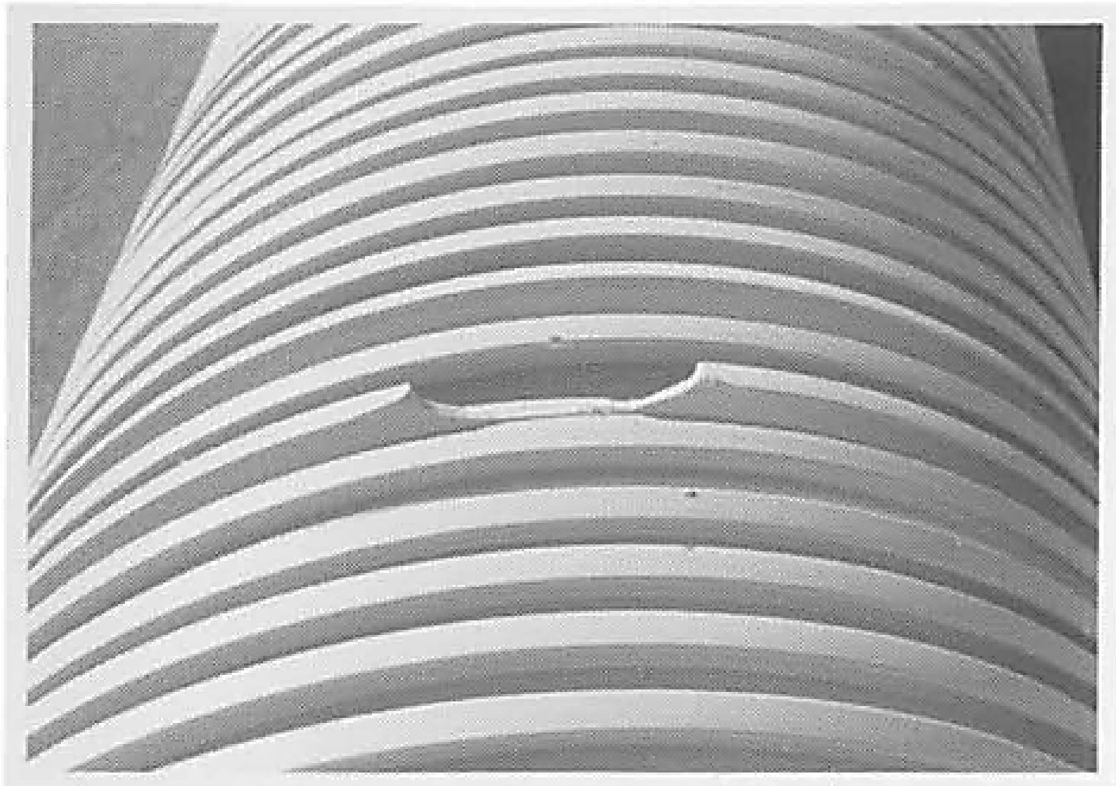


**Figure 5-51: Chipped 21-inch Polyvinyl Chloride Sewer Pipe (Woodham 1988)**

Figure 5-52 illustrates the proper measurement of soil cover above a flexible pipe depending on pavement type.

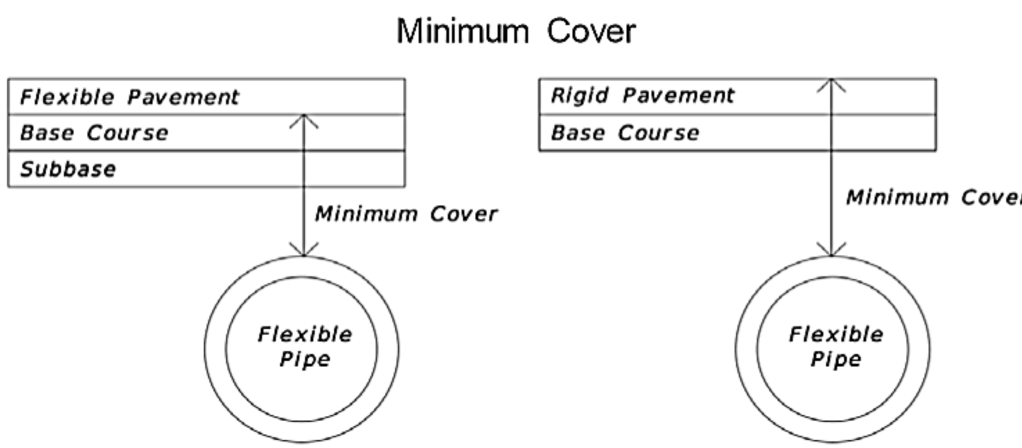



**Figure 5-52: Minimum Cover Height for Plastic Pipes (FDOT 2016)**

The California Department of Transportation sets a maximum cover of 35 feet for PVC pipe ranging from 12 inches to 36 inches in diameter (CALTRANS 2014). Tables 5-82 through 5-86 provide the minimum and/or maximum soil cover for PVC pipe stipulated by various state departments of transportation.

**Table 5-82: FDOT's Minimum Cover for Corrugated Polyvinyl Chloride (FDOT 2016)**

| Pipe Type | Pipe Size (inch) | Minimum Cover (inch) |
|---|---|---|
| Corrugated Polyethylene | 12 – 48 | 24 |
| | 60 | 30 |
| Corrugated Polypropylene | 12 – 48 | 24 |
| | 60 | 30 |
| Corrugated Polyvinyl Chloride | – | 24 |
| Steel Reinforced Polyethylene | – | 12 |

**Table 5-83: FDOT's Maximum Cover for Corrugated Polyvinyl Chloride (FDOT 2016)**

| Diameter (inch) | Maximum Cover (feet) |
|---|---|
| 12 | 42 |
| 15 | 45 |
| 18 | 42 |
| 21 | 41 |
| 24 | 41 |

| | |
|---|---|
| 30 | 40 |
| 36 | 40 |

**Table 5-84: KSDOT's Minimum Cover for Polyvinyl Chloride (KSDOT 2007)**

| PE and PVC Size (inch) | Approx. Min. Cover Required for Axle Load of 18 to 50 Kip (feet) | Approx. Min. Cover Required for Axle Load of 50 to 75 Kip (feet) | Approx. Min. Cover Required for Axle Load of 75 to 110 Kip (feet) | Approx. Min. Cover Required for Axle Load of 110 to 150 Kip (feet) |
|---|---|---|---|---|
| 12 to 36 | 2.0 | 2.5 | 3.0 | 3.0 |
| 42 to 48 | 3.0 | 3.0 | 3.5 | 4.0 |
| 54 to 60 | 3.0 | 3.0 | 3.5 | 4.0 |

**Table 5-85: NMDOT's Maximum Cover for Solid Polyvinyl Chloride (NMDOT 2011)**

| Pipe Size | Min. Cover | Cell Class No. 12454C | | Cell Class No. 12364C | |
|---|---|---|---|---|---|
| Diameter Inches | Inches | Minimum Wall Thickness in Inches | | | |
| | | 0.358 | 0.438 | 0.358 | 0.438 |
| | | Maximum Fill Height in Feet | | | |
| 12 | 12 | 65 | | 69 | |
| 15 | 12 | | 62 | | 66 |

**Table 5-86: NMDOT's Maximum Cover for Ribbed Polyvinyl Chloride (NMDOT 2011)**

| Pipe Size | Min. Cover | Cell Class No. 12454C | Cell Class No. 12364C |
|---|---|---|---|
| Diameter (inches) | (inches) | Maximum Fill Height (feet) | |
| 12 | 12 | 37 | 26 |
| 15 | 12 | 32 | 22 |
| 18 | 12 | 33 | 23 |
| 24 | 12 | 29 | 21 |
| 30 | 12 | 28 | 20 |
| 36 | 12 | 27 | 19 |
| 42 | 12 | 26 | 18 |
| 48 | 12 | 24 | 17 |

## 5.10 Literature Review Summary

The commentary to Section 12 of the *AASHTO LRFD Bridge Design Specifications* recommends the following items as useful information for the design of buried structures and tunnel liners. The literature review was developed from these recommendations.

- Strength and compressibility of foundation materials.
- Chemical characteristics of soil and surface waters, e.g., pH, resistivity, and chloride content of soil and pH, resistivity, and sulfate content of surface water.

- Stream hydrology, e.g., flow rate and velocity, maximum width, allowable headwater depth, and scour potential.
- Performance and condition survey of culverts in the vicinity.

Reinforced concrete pipes are the oldest and most commonly used culvert material in North America. Reinforced concrete pipes have the longest expected service life over all other culvert materials. The service life of a reinforced concrete culvert ends when the reinforcing steel has been exposed or significant cracks begin to form. The most critical factor affecting the service life of a concrete pipe is chemical corrosion.

The service life of a corrugated metal culvert ends when the deterioration reaches the point of perforation. The most critical factors affecting the service life of steel pipe and aluminum pipe are corrosion, abrasion, and gage thickness. Aluminized steel is more resistant to corrosion than galvanized steel.

The most critical factors affecting the service life of plastic pipe culverts are oxidation, ultraviolet radiation, and flammability. Plastic pipe culverts are not affected by chemical or electrochemical corrosion. From the standpoint of the goals of this research, there is not significant difference between HDPE, PVC, and polypropylene.

The most common reason for a culvert to fail is due to a gradual weakening caused by corrosion. Corrosion can occur on both the inside and outside of the culvert. Corrosion most commonly attacks unprotected metal culverts and the reinforcement in concrete pipes. Plastic pipes are unaffected by most inorganic acids, alkalis, and aqueous solutions.

Chloride ions are the most extensively documented contaminant that cause corrosion of the embedded steel in concrete. The embedded steel is more susceptible to corrosion if the concrete cover is inadequate, cracked, or highly permeable. Chloride ions are present in deicing salts and seawater. The degradation is often accelerated in regions where successive freeze-thaw cycles occur.

While a high concentration of sulfates may corrode metal culverts, they are typically more damaging to concrete. Concrete pipes are sufficient to withstand sulfate concentrations of 1,000 ppm or less. The most efficient way to protect against a sulfate attack is to choose a cement with a limited amount of tricalcium aluminate. Resistivity is a measure of the soil's ability to conduct electrical current and, it greatly affects metal culverts. Resistivity values greater than 3,000 ohm-cm are considered high and will impede corrosion.

The pH is an expression of the hydrogen ion concentration in water or soil. It is extremely important that the appropriate culvert material is chosen for the specific site conditions. The FHWA allows high density polyethylene culverts to be used without regard to the resistivity and pH of the soil. However, the same liberties cannot be applied to metal or concrete culverts.

ALDOT has adopted criteria for selecting roadway pipe based upon factors that typically affect the durability, which is reproduced in Table 5-87 (ALDOT Guidelines for Operation 3-22, "Selection of Type of Roadway Pipe," 2017).

Abrasion often accelerates corrosion by stripping away the surface material or protective coating of a culvert. Once the protective coating has been removed, the culvert's main defense against corrosion has been destroyed. Steel pipe is the most susceptible to corrosion. However, aluminum pipe offers no improvement.

Plastic pipe exhibits good abrasion resistance and will likely not experience the dual action of corrosion and abrasion. However, this claim was based on tests using small aggregate sizes flowing at low velocities. The effects of large bedload particles or high velocity flows are not well documented. In addition, rehabilitative strategies have not been specifically developed for plastic pipe due to their more recent emergence as a culvert material.

Plastic pipes are affected by oxidation and ultraviolet radiation. Ultraviolet stabilizers and antioxidant packages are used to protect plastic pipes from this form of degradation. All culvert materials are affected by fire and extremely high temperatures. While it is highly unlikely for concrete to burn, extremely high temperatures can affect the compressive strength, flexural strength, and modulus of elasticity of concrete. Several state departments of transportation have reported that the pipe ends and the flared end sections of polyethylene pipe have caught on fire as a result of crop, leaf, or controlled burning in roadside ditches.

The minimum and maximum fill heights for culverts vary with each department of transportation. The fill height for reinforced concrete pipe is dependent on the D-load, installation type, and pipe diameter. The fill height for standard corrugated metal pipe is dependent on pipe diameter, wall thickness, and corrugations. The fill height for plastic pipe is dependent on pipe material and pipe diameter. Plastic pipe requires a minimum 2 feet of cover.

Table 5-87 presents a summary of material type criteria based upon literature.

Table 5-87: ALDOT Criteria for "Selection of Type of Roadway Pipe"

| Pipe Type | pH | Resistivity | Chloride | Sulfate | Abrasion Resistance |
|---|---|---|---|---|---|
| Galvanized steel or Aluminum | >10 | >3000Ωcm | <50 mg/L | <100 mg/L | Mild |
| Bituminous Coated Galvanized Steel or Aluminum | 5-10 | >3000Ωcm | <50 mg/L | <100 mg/L | Moderate |
| Bituminous Coated Galvanized Steel with Paved Invert | 5-10 | >3000Ωcm | <50 mg/L | <100 mg/L | Moderate to Good |
| Aluminized Steel | 5-9 | >1500Ωcm | <50 mg/L | <100 mg/L | Moderate |
| Plain Concrete or Reinforced Concrete | May be used in all situations. If pH is below 4, a special coating is required. | | | | |

**Table 5-88: Summarized Comparison of Culvert Material Types from Literature**

| | Concrete | Steel | | Aluminum | Plastic |
|---|---|---|---|---|---|
| | | Galvanized | Aluminized | | |
| Sulfate Concentration | Susceptible to degradation if the sulfate is ≥1,000 ppm | Susceptible to degradation if the sulfate is ≥1,000 ppm | Susceptible to degradation if the sulfate is ≥1,000 ppm | Susceptible to degradation if the sulfate is ≥1,000 ppm | Not Applicable |
| Resistivity | May require additional cover over embedded reinforcing steel | Susceptible to degradation if the resistivity is ≤3,000 ohm-cm | Susceptible to degradation if the resistivity is ≤3,000 ohm-cm | Susceptible to degradation if the resistivity is ≤3,000 ohm-cm | Not Applicable |
| pH Concentration (acceptable range) | pH > 5 | 6.0 < pH < 10 | 5.0 < pH < 9.0 | 5.0 < pH < 9.0 | Not Applicable |
| Chloride Concentration (acceptable range) | < 50 mg/L | < 50 mg/L | < 50 mg/L | < 50 mg/L | Not Applicable |
| Abrasion | Modify mix design for severe abrasion | Not recommended for moderate or severe abrasion | Not recommended for moderate or severe abrasion | Not recommended for moderate or severe abrasion | Not recommended for severe abrasion |
| Ultraviolet Radiation | Not Applicable | Not Applicable | Not Applicable | Not Applicable | Most susceptible to degradation |
| Flammability / Heat | Least susceptible | Protective coatings may burn | Protective coatings may burn | Protective coatings may burn | Most susceptible |
| Maximum Pipe Diameter | 96 inch | 72 inch | 72 inch | 72 inch | 48 inch |
| Minimum Fill Height | 1 – 2 feet | 1 – 2 feet | 1 – 2 feet | 1 – 2 feet | 2 feet |
| Maximum Fill Height | Not as limited | Not as limited | Not as limited | Not as limited | 20 feet |
| Quality-Controlled Installation | Not as critical | Important | Important | Important | Required |

# SECTION 6

# DECISION ALGORITHM

## 6.1 Introduction

*The Specification for Culvert Material Selection* presented in Appendix B provides guidance and the latest research to aid in the selection of culvert material for cross-drainage application. It is comprised of checklists and a condensed summary of the material contained within this report. The checklists facilitate a quick selection or elimination of culvert material based upon site conditions. The accompanying condensed summary will serve as an on-the-go guide that represents fundamental information that has evolved from this research. The methodology and parameter limits are largely based upon the information presented in Section 5 and therefore some aspects of the approach may need to be adjusted to fit specific ALDOT needs and requirements.

The checklists were created from crucial durability concerns and installation requirements and are applicable only to trench installations. The checklists may be completed either by hand or by electronic methods. Crucial durability concerns include: sulfate content, resistivity, pH levels, chloride concentration, abrasion, flammability, and ultraviolet radiation. Installation requirements include: minimum soil cover, maximum soil cover, culvert diameter, and presence of quality control personnel.

The following flowcharts illustrate the decision process and pertinent information that was used to create *The Specification for Culvert Material Selection*, which was developed from these flowcharts and by the input parameters defined in *Cross-Drain Pipe Material Selection Algorithm* (French 2013).

### 6.2 Flowchart Key

Table 6-1 provides the abbreviations that were used to create the flowcharts. Table 6-2 depicts the three shapes were chosen to create the flowcharts. These three shapes will be used throughout each flowchart to maintain consistency. The colors were only chosen to visually assist the user. Therefore, any of the material may be printed in black and white without the concern of losing vital information.

**Table 6-1: Flowchart Abbreviations**

| | |
|---|---|
| RCP | Reinforced Concrete Pipe |
| CSP | Corrugated Steel Pipe |
| CAP | Corrugated Aluminum Pipe |
| P | Plastic |

**Table 6-2: Flowchart Key**

| | |
|---|---|
|  | Decision – This shape will pose a question to the user. |
|  | Answer – This shape will show the recommended culvert material. |
|  | Reroute – This shape may direct the user to a different flowchart. |

### 6.3 Selection Process

The selection process begins by determining the required service life of the culvert. Reinforced concrete culverts have the longest expected service life over all other culvert materials. Therefore, reinforced concrete culverts are recommended when the required service life exceeds 75 years. If the service life of the culvert is not expected to exceed 75 years, then reinforced concrete, corrugated steel, corrugated aluminum, and plastic are all acceptable culvert materials.

If the required service life exceeds 75 years, a checklist is provided to ensure environmental and site conditions are suitable for a reinforced concrete pipe. If the service life of the culvert is not expected to exceed 75 years, the next deciding factor is the pipe diameter.

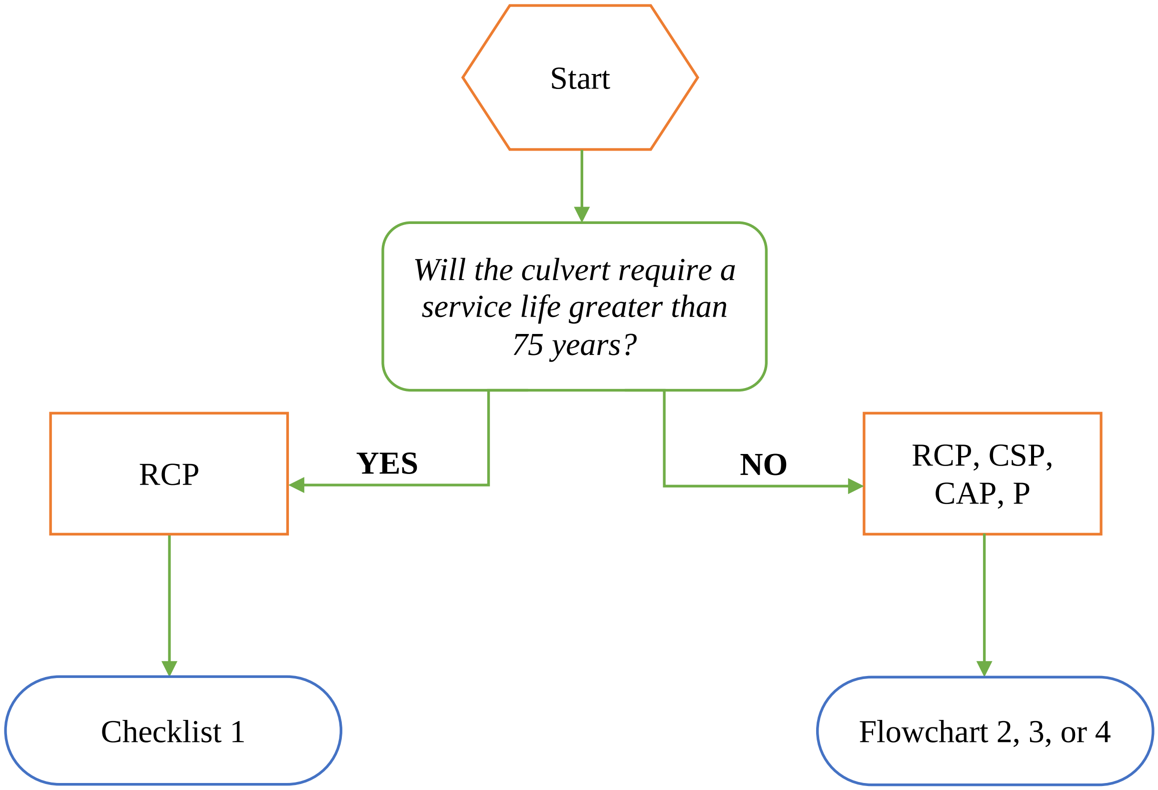



**Figure 6-1: Flow Chart 1**

The largest plastic pipe diameter that will be considered is 48 inches since most state departments of transportation are hesitant to allow plastic pipes of greater diameter. If the design requires a pipe diameter larger than 48 inches, plastic pipes are no longer considered acceptable culvert materials. Reinforced concrete, corrugated steel, and corrugated aluminum are recommended when the pipe diameter exceeds 48 inches. A checklist is provided to ensure environmental and site conditions are suitable for a corrugated metal pipe. It should be noted however that ALDOT currently allows plastic pipe up to 54 inches in diameter.

After the required service life and the required pipe diameter have been established, the acceptable culvert materials are narrowed down based on environmental and site conditions.

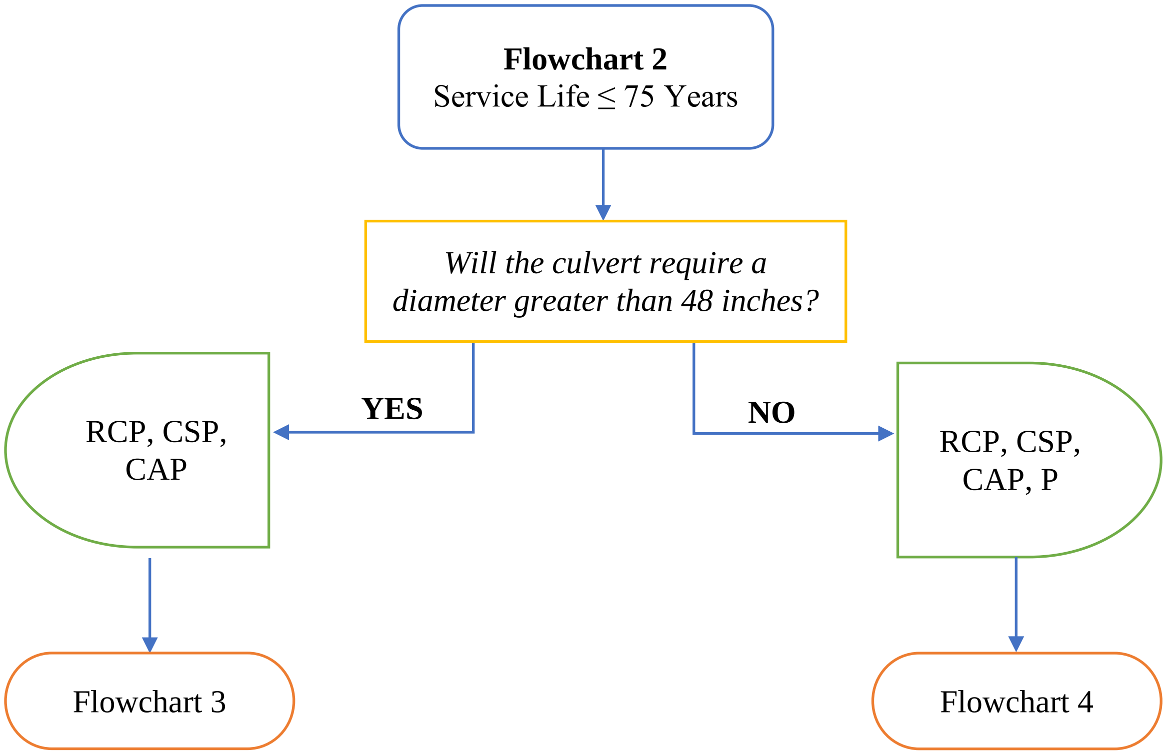



**Figure 6-2: Flow Chart 2**

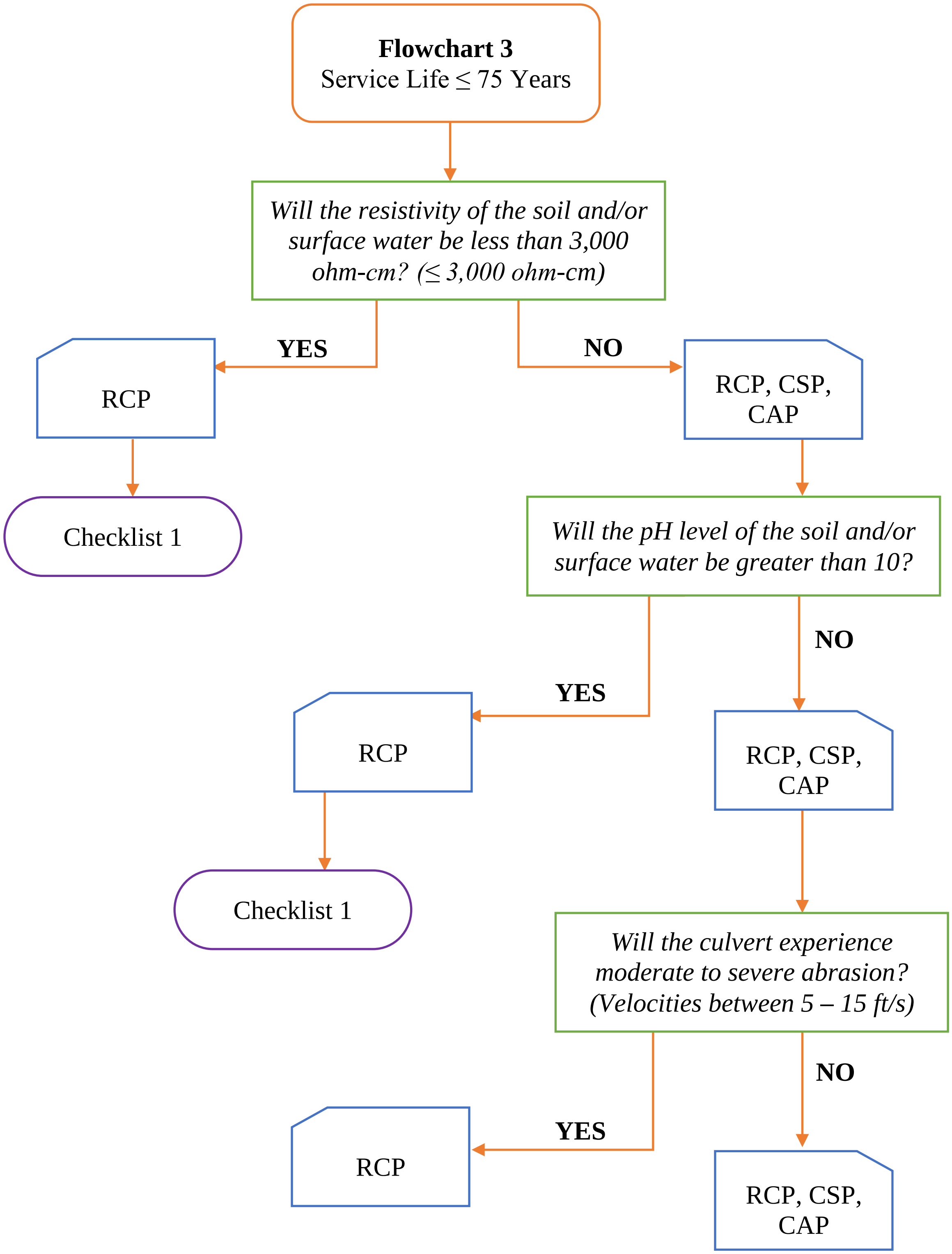



**Figure 6-3: Flow Chart 3**

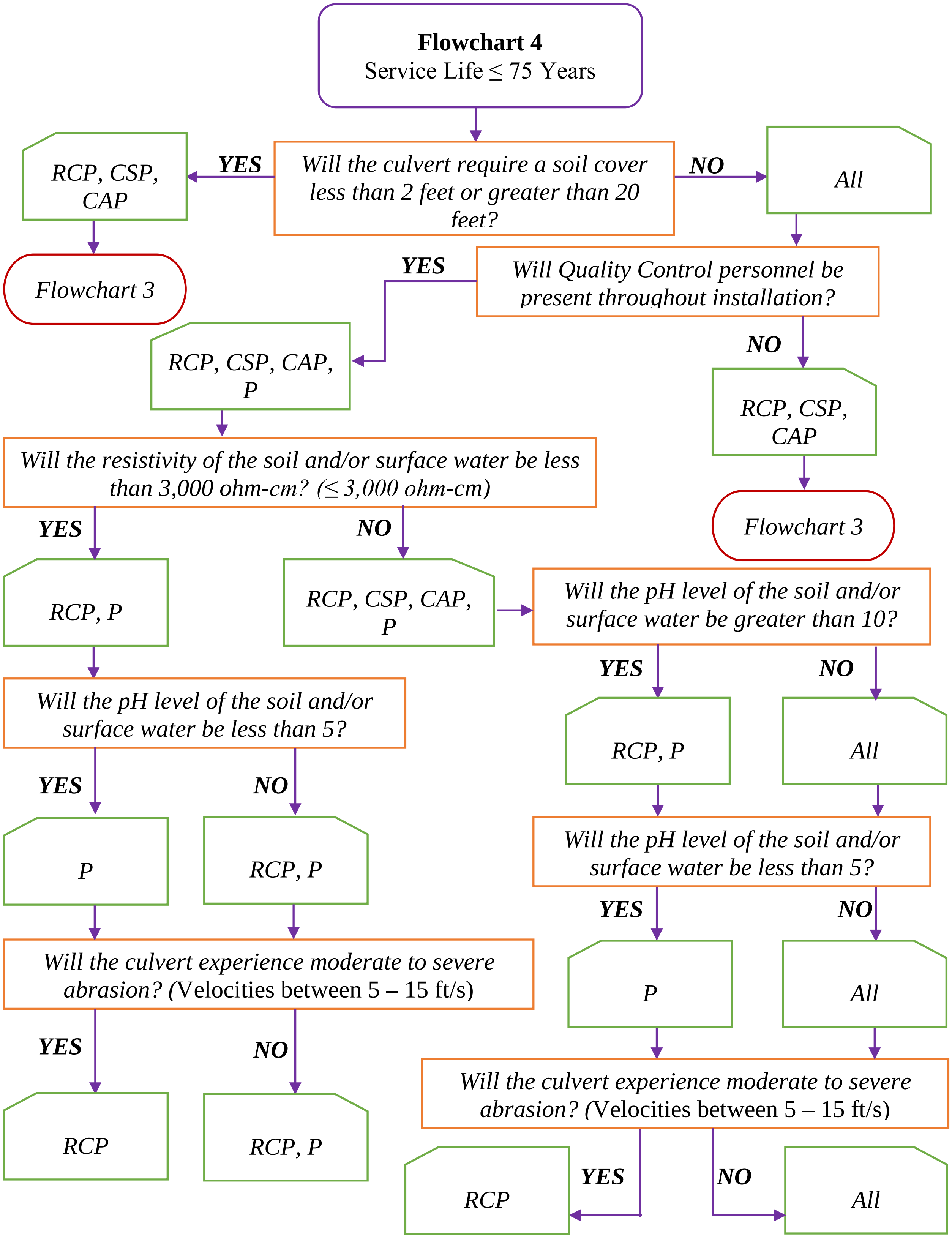



**Figure 6-4: Flow Chart 4**

## Checklist 1: Concrete Culverts

Directions: Place a check in the box if the condition is true. Note if additional protection is required. A reinforced concrete culvert should only be used if the provisions of Checklist 1 are satisfied.

| | | | |
|---|---|---|---|
| **Sulfate Content** | | | |
| Will the sulfate content of the surface water be less than 1,000 ppm? (<1,000 ppm) | | ☐ | No additional protection required. |
| Will the sulfate content of the surface water exceed 1,000 ppm? (≥1,000 ppm) | | ☐ | Additional protection required. |
| **Resistivity** | | | |
| Will the resistivity of the soil and/or surface water be less than 3,000 ohm-cm? (≤ 3,000 ohm-cm) | | ☐ | Ensure adequate cover over reinforcing steel. |
| Will the resistivity of the soil and/or surface water exceed 3,000 ohm-cm? (> 3,000 ohm-cm) | | ☐ | No additional protection required. |
| **pH Level** | | | |
| Will the pH of the soil and/or surface water be less than 5? | | ☐ | Reinforced concrete should not be used. |
| Will the pH of the soil and/or surface water exceed 5? | | ☐ | No additional protection required. |
| **Abrasion** | | | |
| Level 1 | Non-abrasive conditions exist in areas of no bed load and very low velocities. | ☐ | No additional protection required. |
| Level 2 | Low abrasive conditions exist in areas of minor bed loads of sand and velocities of 5 feet/second or less | ☐ | No additional protection required. |
| Level 3 | Moderate abrasive conditions exists in areas of moderate bed loads of sand and gravel and velocities between 5 feet/second and 15 feet/second | ☐ | No additional protection required. |
| Level 4 | Severe abrasive conditions exist in areas of heavy bed loads of sand, gravel and rock and velocities exceeding 15 feet/second | ☐ | Additional protection required. |

## Checklist 2: Metal Culverts

Directions: Place a check in the box if the condition is true. Note if additional protection is required. A metal culvert should only be used if the provisions of Checklist 2 are satisfied.

| | | | |
|---|---|---|---|
| **Service Life** | | | |
| Will the service life of the culvert exceed 75 years? | | ☐ | Metal culverts should not be used. |
| **Sulfate Content** | | | |
| Will the sulfate content of the surface water be less than 1,000 ppm? (<1,000 ppm) | | ☐ | No additional protection required. |
| Will the sulfate content of the surface water exceed 1,000 ppm? (≥1,000 ppm) | | ☐ | Additional protection required. |
| **Resistivity** | | | |
| Will the resistivity of the soil and/or surface water be less than 3,000 ohm-cm? (≤ 3,000 ohm-cm) | | ☐ | Metal culverts should not be used. |
| Will the resistivity of the soil and/or surface water exceed 3,000 ohm-cm? (> 3,000 ohm-cm) | | ☐ | No additional protection required. |
| **pH Level** | | | |
| 6.0 < pH < 10 | | ☐ | Galvanized steel culverts. |
| 5.0 < pH < 9.0 | | ☐ | Aluminum culverts. |
| **Abrasion** | | | |
| Level 1 | Non-abrasive conditions exist in areas of no bed load and very low velocities. | ☐ | No additional protection required. |
| Level 2 | Low abrasive conditions exist in areas of minor bed loads of sand and velocities of 5 feet/second or less | ☐ | No additional protection required. |
| Level 3 | Moderate abrasive conditions exists in areas of moderate bed loads of sand and gravel and velocities between 5 feet/second and 15 feet/second | ☐ | Metal culverts should not be used. |
| Level 4 | Severe abrasive conditions exist in areas of heavy bed loads of sand, gravel and rock and velocities exceeding 15 feet/second | ☐ | Metal culverts should not be used. |

## Checklist 3: Plastic Culverts

Directions: Place a check in the box if the condition is true. Note if additional protection is required. A plastic culvert should only be used if the provisions of Checklist 3 are satisfied.

| | | | |
|---|---|---|---|
| **Service Life** | | | |
| Will the service life of the culvert exceed 75 years? | | ☐ | Plastic culverts should not be used. |
| **Installation** | | | |
| Will the culvert require a diameter greater than 48 inches? | | ☐ | Plastic culverts should not be used. |
| Will the culvert require a minimum fill height less than 2 feet? | | ☐ | Plastic culverts should not be used. |
| Will the culvert require a maximum fill height greater than 20 feet? | | ☐ | Plastic culverts should not be used. |
| **Quality Control** | | | |
| Will quality control personnel be present throughout installation? | | ☐ | If not, plastic culverts should not be used. |
| **Ultraviolet Radiation** | | | |
| Will the culvert be exposed to prolonged sunlight/ ultraviolet radiation? | | ☐ | Additional protection required. |
| **Flammability** | | | |
| Will the culvert be installed at a location prone to fires? | | ☐ | Additional protection required. |
| **Sulfate Content, Resistivity, pH Level** | | | |
| No limitation. | | | |
| **Abrasion** | | | |
| Level 1 | Non-abrasive conditions exist in areas of no bed load and very low velocities. | ☐ | No additional protection required. |
| Level 2 | Low abrasive conditions exist in areas of minor bed loads of sand and velocities of 5 feet/second or less | ☐ | No additional protection required. |
| Level 3 | Moderate abrasive conditions exists in areas of moderate bed loads of sand and gravel and velocities between 5 feet/second and 15 feet/second | ☐ | No additional protection required. |
| Level 4 | Severe abrasive conditions exist in areas of heavy bed loads of sand, gravel and rock and velocities exceeding 15 feet/second | ☐ | Plastic culverts should not be used. |

<u>CULVERT PIPE SELECTION WORKSHEET</u>

*This chart was developed for preliminary guidance on the selection of manufactured pipe types that can be used for a given cross drain culvert application. As such, the criteria are very general and are not intended to replace design specifications and limitations. Click the box if the answer is "yes"; all boxes must be checked for the pipe type to be acceptable.*

| **Precast Concrete Pipe** | |
|---|---|
| Will the **sulfate content** of the surface water be less than 1000 ppm? If "no" then additional coating/protection may be required. | ☐ |
| Will the **resistivity** of the soil and/or surface water be greater than 3000 ohm-cm? If "no" then sufficient cover over reinforcing steel must be ensured. | ☐ |
| Will the **pH** of the soil and/or surface water be greater than 4? If "no" then additional coating / protection may be required. | ☐ |

| **Metal Pipe** | |
|---|---|
| Will the **service life** of the culvert be less than 75 years? | ☐ |
| Will the **sulfate content** of the soil and/or surface water be less than 1000 ppm and 100 mg/L? | ☐ |
| Will the **resistivity** of the soil and/or surface water be greater than: 3000 ohm-cm (for galvanized steel or aluminum) or 1500 ohm-cm (for aluminum and aluminized steel)? | ☐ |
| Will the **pH** level be: 5 < pH < 10 (for galvanized steel or aluminum), 5 < pH < 9 (for aluminum and aluminized steel)? | ☐ |
| Will the **chloride concentration** of the soil and/or surface water be less than 50 mg/L? | ☐ |
| Will the **ADT** be less than 4000? | ☐ |
| Will the pipe meet the Level 2 **abrasion** definition? If "no" then additional coating / protection may be required. | ☐ |

| **Plastic Pipe** | |
|---|---|
| Will the **service life** of the culvert be less than 75 years? | ☐ |
| Will the required pipe **diameter** be less than 54 inches? | ☐ |
| Will the **fill height** be greater than 3 feet but less than 20 feet? | ☐ |
| Will **quality control** personnel rigorously monitor the entire installation? | ☐ |
| Will the culvert be fully sheltered from prolonged sunlight / **ultraviolet radiation**? | ☐ |
| Will there be no potential that the pipe will be subjected to **fire or extreme heat**? | ☐ |
| Will the **ADT** be less than 4000? | ☐ |
| Will the pipe meet the Level 3 **abrasion** definition? | ☐ |

# SECTION 7

# PIPE SELECTION DEMONSTRATION

## 7.1 Demonstration

The following culvert installation projects have been selected to illustrate the usefulness and accuracy of *The Specification for Culvert Material Selection*. The bases of these demonstrations were actual installations, but when specific project information was not available, reasonable parameters were adopted. Four demonstrations were created to represent culvert installation projects throughout the United States. Installation consisted of a new circular culvert. Rehabilitation projects were not considered.

## 7.2 Interstate 81 (Pennsylvania)

This Pennsylvania Department of Transportation installation was of a new culvert beneath Interstate 81 in Carlisle. According to design calculations, the culvert will be 24-inches in diameter, and require a minimum fill height of 1 foot and a maximum burial depth of 12 feet. The culvert must be designed to last 50 years. The area is prone to heavy snowfall and does not experience severely high temperatures. Extensive environmental testing was performed and the results are shown below. Representatives of the Pennsylvania Department of Transportation have confirmed that quality control personnel were present throughout installation. Fire and long-term ultraviolet radiation exposure is unlikely.

**Table 7-1: Environmental Testing Results for Interstate 81**

| Sulfate Content | pH Concentration | Resistivity | Velocity |
|---|---|---|---|
| 570 ppm | 6 | 6,000 ohm-cm | 5 – 10 feet/second |


**Figure 7-1: Location of Proposed Culvert Installation beneath Interstate 81**

Based on the following chart, the Pennsylvania Department of Transportation can choose from three possible culvert material types. A reinforced concrete culvert will meet all of the project criteria and not require additional protection. A corrugated aluminized steel culvert and a corrugated aluminum culvert will also meet all of the project criteria but will require additional protection (i.e., increased gage thickness) to protect against abrasion. Plastic culvert was eliminated due to minimum fill height requirements.

Directions: Input the predetermined site/project specific conditions into the blank column shown. Compare the inputted data to the data shown to the right of the bolded black line. Note if additional protection is required.

| | Site Conditions | Reinforced Concrete | Galvanized Steel | Aluminized Steel | Aluminum | Plastic |
|---|---|---|---|---|---|---|
| **Service Life (Years)** | 50 | + 75 | 75 | 75 | 75 | 75 |
| **Sulfate Content (ppm)** | 570 | ≤ 1,000 [1] | ≤ 1,000[1] | ≤ 1,000[1] | ≤ 1,000[1] | N/A |
| **Resistivity (ohm-cm)** | 6,000 | > 3,000[2] | > 3,000[2] | > 3,000[2] | > 3,000[2] | N/A |
| **pH Levels** | 6 | pH < 5 | 6 < pH < 10 | 5 < pH < 9 | 5 < pH < 9 | N/A |
| **Abrasion (Level)** | 3 | 1, 2, 3 | 1, 2, 3[3] | 1, 2, 3[3] | 1, 2, 3[3] | 1, 2, 3 |
| **Ultraviolet Radiation** | No | N/A | N/A | N/A | N/A | Highly Sensitive |
| **Flammability/ Heat** | No | N/A | Sensitive | Sensitive | Sensitive | Highly Sensitive |
| **Max. Pipe Diameter (inch)** | 24 | 96 | 72 | 72 | 72 | 48 |
| **Min. Fill Height (feet)** | 1 | 1 – 2 | 1 – 2 | 1 – 2 | 1 – 2 | 2 |
| **Max. Fill Height (feet)** | 12 | N/A | N/A | N/A | N/A | 20 |
| **Quality Controlled Installation** | Yes | N/A | Sensitive | Sensitive | Sensitive | Highly Sensitive |

Note:
(1) Additional protection required if the sulfate content exceeds 1,000 ppm.
(2) Additional protection required if the resistivity is less than 3,000 ohm-cm. Metal culverts not recommended.
(3) Additional protection required for moderate abrasion conditions.

### 7.3 Interstate 520 (Georgia)

The Georgia Department of Transportation has begun the design of a new culvert beneath Interstate 520 in Augusta. The culvert will be 30 inches in diameter, and require a minimum fill height of 2 feet and a maximum burial depth of 20 feet. The culvert must be designed to last 70 years. The area is not prone to heavy snowfall and experiences relatively high temperatures. Extensive environmental testing has been performed and the results are shown below. Quality control personnel will be present throughout installation. Fire and long-term ultraviolet radiation exposure is unlikely.

**Table 7-2: Environmental Testing Results for Interstate 520**

| Sulfate Content | pH Concentration | Resistivity | Velocity |
|---|---|---|---|
| 850 ppm | 9 | 10,000 ohm-cm | 5 feet/second |


**Figure 7-2: Location of Proposed Culvert Installation beneath Interstate 520**

Based on the following chart, the Georgia Department of Transportation can choose from reinforced concrete culvert, corrugated galvanized steel culvert, and plastic pipes.

Directions: Input the predetermined site/project specific conditions into the blank column shown. Compare the inputted data to the data shown to the right of the bolded black line. Note if additional protection is required.

| | Site Conditions | Reinforced Concrete | Galvanized Steel | Aluminized Steel | Aluminum | Plastic |
|---|---|---|---|---|---|---|
| **Service Life (Years)** | 70 | + 75 | 75 | 75 | 75 | 75 |
| **Sulfate Content (ppm)** | 850 | ≤ 1,000 [1] | ≤ 1,000[1] | ≤ 1,000[1] | ≤ 1,000[1] | N/A |
| **Resistivity (ohm-cm)** | 10,000 | > 3,000[2] | > 3,000[2] | > 3,000[2] | > 3,000[2] | N/A |
| **pH Levels** | 9 | pH < 5 | 6 < pH < 10 | 5 < pH < 9 | 5 < pH < 9 | N/A |
| **Abrasion (Level)** | 2 | 1, 2, 3 | 1, 2, 3[3] | 1, 2, 3[3] | 1, 2, 3[3] | 1, 2, 3 |
| **Ultraviolet Radiation** | No | N/A | N/A | N/A | N/A | Highly Sensitive |
| **Flammability/ Heat** | No | N/A | Sensitive | Sensitive | Sensitive | Highly Sensitive |
| **Max. Pipe Diameter (inch)** | 30 | 96 | 72 | 72 | 72 | 48 |
| **Min. Fill Height (feet)** | 2 | 1 – 2 | 1 – 2 | 1 – 2 | 1 – 2 | 2 |
| **Max. Fill Height (feet)** | 20 | N/A | N/A | N/A | N/A | 20 |
| **Quality Controlled Installation** | Yes | N/A | Sensitive | Sensitive | Sensitive | Highly Sensitive |

Note:
(1) Additional protection required if the sulfate content exceeds 1,000 ppm.
(2) Additional protection required if the resistivity is less than 3,000 ohm-cm. Metal culverts not recommended.
(3) Additional protection required for moderate abrasion conditions.

### 7.4 Route 25 (South Carolina)

The South Carolina Department of Transportation plans to install a new culvert beneath Route 25 in Greenville. The culvert will be 18-inches in diameter, and require a minimum fill height of 2 feet and a maximum burial depth of 40 feet. The culvert is expected to last 75 years. The area is not prone to heavy snowfall and experiences relatively high temperatures. Extensive environmental testing has been performed and the results are shown below. Quality control personnel will be present throughout installation. Fire is possible at this location.

**Table 7-3: Environmental Testing Results for Route 25**

| Sulfate Content | pH Concentration | Resistivity | Velocity |
|---|---|---|---|
| 1,200 ppm | 8 | 4,000 ohm-cm | 5 feet/second |


**Figure 7-3: Location of Proposed Culvert Installation beneath Route 25**

Based on the following chart, the South Carolina Department of Transportation will need to provide additional protection. Plastic pipes satisfy all of the requirements with the exception of the maximum fill height. Reinforced concrete culverts, corrugated steel culverts, and corrugated aluminum culverts are susceptible to degradation when the sulfate content is greater than 1,000 ppm. Therefore, if the maximum fill height cannot be modified, a reinforced concrete culvert, a corrugated steel culvert, and a corrugated aluminum culvert are all acceptable material types on the condition that additional protection is provided.

Directions: Input the predetermined site/project specific conditions into the blank column shown. Compare the inputted data to the data shown to the right of the bolded black line. Note if additional protection is required.

| | Site Conditions | Reinforced Concrete | Galvanized Steel | Aluminized Steel | Aluminum | Plastic |
|---|---|---|---|---|---|---|
| **Service Life (Years)** | 75 | + 75 | 75 | 75 | 75 | 75 |
| **Sulfate Content (ppm)** | 1,200 | ≤ 1,000 [(1)] | ≤ 1,000[(1)] | ≤ 1,000[(1)] | ≤ 1,000[(1)] | N/A |
| **Resistivity (ohm-cm)** | 4,000 | > 3,000[(2)] | > 3,000[(2)] | > 3,000[(2)] | > 3,000[(2)] | N/A |
| **pH Levels** | 8 | pH < 5 | 6 < pH < 10 | 5 < pH < 9 | 5 < pH < 9 | N/A |
| **Abrasion (Level)** | 2 | 1, 2, 3 | 1, 2, 3[(3)] | 1, 2, 3[(3)] | 1, 2, 3[(3)] | 1, 2, 3 |
| **Ultraviolet Radiation** | No | N/A | N/A | N/A | N/A | Highly Sensitive |
| **Flammability/ Heat** | No | N/A | Sensitive | Sensitive | Sensitive | Highly Sensitive |
| **Max. Pipe Diameter (inch)** | 18 | 96 | 72 | 72 | 72 | 48 |
| **Min. Fill Height (feet)** | 2 | 1 – 2 | 1 – 2 | 1 – 2 | 1 – 2 | 2 |
| **Max. Fill Height (feet)** | 40 | N/A | N/A | N/A | N/A | 20 |
| **Quality Controlled Installation** | Yes | N/A | Sensitive | Sensitive | Sensitive | Highly Sensitive |

Note:
[(1)] Additional protection required if the sulfate content exceeds 1,000 ppm.
[(2)] Additional protection required if the resistivity is less than 3,000 ohm-cm. Metal culverts not recommended.
[(3)] Additional protection required for moderate abrasion conditions.

### 7.5 Robinson Road (Mississippi)

The Mississippi Department of Transportation has begun the design of a new culvert beneath Robinson Road in Jackson. The culvert will be 36-inches in diameter, and require a minimum fill height of 2 feet and a maximum burial depth of 15 feet. The culvert must last 70 years. The area is not prone to snowfall and experiences relatively high temperatures. Extensive environmental testing has been performed and the results are shown below. Quality control personnel will not be present during installation. The likelihood of fire is low.

**Table 7-4: Environmental Testing Results for Robinson Road**

| Sulfate Content | pH Concentration | Resistivity | Velocity |
|---|---|---|---|
| 700 ppm | 5 | 3,500 ohm-cm | 5 to 15 feet/second |


**Figure 7-4: Location of Proposed Culvert Installation beneath Robinson Road**

Based on the following chart, the Mississippi Department of Transportation can only choose a reinforced concrete culvert.  A corrugated aluminized steel culvert, a corrugated galvanized steel culvert, and a corrugated aluminum culvert were eliminated due to pH levels.  A plastic culvert was eliminated due to the lack of presence of quality control personnel.

Directions: Input the predetermined site/project specific conditions into the blank column shown. Compare the inputted data to the data shown to the right of the bolded black line. Note if additional protection is required.

| | Site Conditions | Reinforced Concrete | Galvanized Steel | Aluminized Steel | Aluminum | Plastic |
|---|---|---|---|---|---|---|
| **Service Life (Years)** | 65 | + 75 | 75 | 75 | 75 | 75 |
| **Sulfate Content (ppm)** | 700 | ≤ 1,000 [1] | ≤ 1,000[1] | ≤ 1,000[1] | ≤ 1,000[1] | N/A |
| **Resistivity (ohm-cm)** | 3,500 | > 3,000[2] | > 3,000[2] | > 3,000[2] | > 3,000[2] | N/A |
| **pH Levels** | 5 | pH < 5 | 6 < pH < 10 | 5 < pH < 9 | 5 < pH < 9 | N/A |
| **Abrasion (Level)** | 3 | 1, 2, 3 | 1, 2, 3[3] | 1, 2, 3[3] | 1, 2, 3[3] | 1, 2, 3 |
| **Ultraviolet Radiation** | No | N/A | N/A | N/A | N/A | Highly Sensitive |
| **Flammability/ Heat** | No | N/A | Sensitive | Sensitive | Sensitive | Highly Sensitive |
| **Max. Pipe Diameter (inch)** | 36 | 96 | 72 | 72 | 72 | 48 |
| **Min. Fill Height (feet)** | 2 | 1 – 2 | 1 – 2 | 1 – 2 | 1 – 2 | 2 |
| **Max. Fill Height (feet)** | 15 | N/A | N/A | N/A | N/A | 20 |
| **Quality Controlled Installation** | No | N/A | Sensitive | Sensitive | Sensitive | Highly Sensitive |

Note:
[1] Additional protection required if the sulfate content exceeds 1,000 ppm.
[2] Additional protection required if the resistivity is less than 3,000 ohm-cm. Metal culverts not recommended.
[3] Additional protection required for moderate abrasion conditions.

# SECTION 8

# TESTING PROTOCOLS BACKGROUND

## 8.1 Introduction

The use of thermoplastic pipe has increased in recent years (Gassman, Schroeder, and Ray 2005). However, thermoplastic pipe is still a relatively new product that many civil engineers do not feel comfortable with relative to steel and concrete pipes, especially, when it comes to the long term performance of the thermoplastic pipe (Sargand et al. 2002). The Federal Highway Administration expanded the list of acceptable culvert materials that could be specified on federally-aided projects and all the listed pipes must be equally considered (Civil + Structural Engineer 2006). Thermoplastic pipes have many benefits that make them a credible choice including that they are lighter, more cost-effective, and more resistant to chemical attacks than most of the conventional pipes (Sargand et al. 2002).

Auburn University began a study in 2008 to monitor the installation and performance of thermoplastic pipe under beehive road in Lee County, Alabama where exits and on-ramps for I-85 were added. Five lines of plastic pipes totaling over 1000 feet were installed for long-term evaluation purposes. It took three years for the initial performance data to be collected for the pipe due to a redesign to accommodate the plastic pipes that caused an increase design cost and construction time (Stuart et al. 2011).

This section of the report provides a review of plastic pipe in respect to materials and pipe-soil interaction. It also introduces other testing apparatus, past experiments, and current test performed to qualify plastic pipe.

**8.2 Flexible Pipe**

The two main plastic pipe materials that are being considered in this section are high-density polyethylene (HDPE) and polyvinyl chloride (PVC), although a third option, polypropylene (PP), has recently entered the U.S. market. Thermoplastic pipes are made of nonlinear viscoelastic materials. They have a nonlinear and time-dependent response to stress, unlike steel that has a linear response to stress until a reaching a relatively high yield point. Thermoplastic pipes have problems with creep and stress relaxation that causes more deflection over time. (PPI 2008)

**8.3 Soil/Pipe Interaction**

The condition and properties of the soil surrounding the pipe becomes vital to the performance of the plastic pipe. When the load is applied, the pipe will deflect and the loads will be transferred to the surrounding soil as illustrated in Figure 8-1. Since the pipe is flexible, it is capable of arching without giving up its structural integrity (PPI 2008).

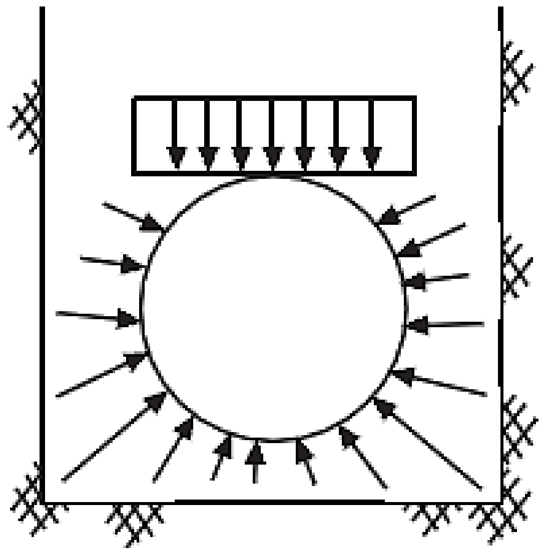


**Figure 8-1: Load Transfer for Flexible Pipe (PPI 2008)**

8.3.1 Spangler's Modified Iowa Formula

The deflection of flexible pipe is typically calculated using the Spangler's Modified Iowa Formula. It measures the deflection of the pipe within the pipe soil reaction and can be seen in Figure 8-2. One of the most important parameters is the modulus of soil reaction (E') that represents the support from the soil surrounding the pipe. Typical E' values can be seen in Figure 8-3 and Figure 8-4 (PPI 2008).

(3-10)

$$\frac{\Delta X}{D_M} = \frac{1}{144}\left(\frac{K_{BED}\, L_{DL}\, P_E + K_{BED} P_L}{\frac{2E}{3}\left(\frac{1}{DR-1}\right)^3 + 0.061 F_S E'}\right)$$

and for use with ASTM F894 profile wall pipe as:

(3-11)

$$\frac{\Delta X}{D_I} = \frac{P}{144}\left(\frac{K_{BED} L_{DL}}{\frac{1.24(RSC)}{D_M} + 0.061 F_S E'}\right)$$

**WHERE**

$\Delta X$ = Horizontal deflection, in

$K_{BED}$ = Bedding factor, typically 0.1

$L_{DL}$ = Deflection lag factor

$P_E$ = Vertical soil pressure due to earth load, psf

$P_L$ = Vertical soil pressure due to live load, psf

$E$ = Apparent modulus of elasticity of pipe material, lb/in$^2$

$E'$ = Modulus of Soil reaction, psi

$F_S$ = Soil Support Factor

$RSC$ = Ring Stiffness Constant, lb/ft

$DR$ = Dimension Ratio, OD/t

$D_M$ = Mean diameter ($D_I$+2z or $D_O$-t), in

$z$ = Centroid of wall section, in

$t$ = Minimum wall thickness, in

$D_I$ = pipe inside diameter, in

$D_O$ = pipe outside diameter, in

**Figure 8-2: Modified Iowa Formula (PPI 2008)**

| Soil Type-pipe Embedment Material (Unified Classification System)[1] | E' for Degree of Embedment Compaction, lb/in$^2$ | | | |
|---|---|---|---|---|
| | Dumped | Slight, <85% Proctor, <40% Relative Density | Moderate, 85%-95% Proctor, 40%-70% Relative Density | High, >95% Proctor, >70% Relative Density |
| Fine-grained Soils (LL > 50)[2] Soils with medium to high plasticity; CH, MH, CH-MH | No data available: consult a competent soils engineer, otherwise, use E' = 0. | | | |
| Fine-grained Soils (LL < 50) Soils with medium to no plasticity, CL, ML, ML-CL, with less than 25% coarse grained particles. | 50 | 200 | 400 | 1000 |
| Fine-grained Soils (LL < 50) Soils with medium to no plasticity, CL, ML, ML-CL, with more than 25% coarse grained particles; Coarse-grained Soils with Fines, GM, GC, SM, SC[3] containing more than 12% fines. | 100 | 400 | 1000 | 2000 |
| Coarse-grained soils with Little or No Fines GW, GP, SW, SP[3] containing less than 12% fines | 200 | 1000 | 2000 | 3000 |
| Crushed Rock | 1000 | 3000 | 3000 | 3000 |
| Accuracy in Terms of Percentage Deflection[4] | ±2% | ±2% | ±1% | ±0.5% |

[1] ASTM D-2487, USBR Designation E-3
[2] LL = Liquid Limit
[3] Or any borderline soil beginning with one of these symbols (i.e., GM-GC, GC-SC).
[4] For ±1% accuracy and predicted deflection of 3%, actual deflection would be between 2% and 4%.

**Note:** Values applicable only for fills less than 50 ft (15 m). Table does not include any safety factor. For use in predicting initial deflections only; appropriate Deflection Lag Factor must be applied for long-term deflections. If embedment falls on the borderline between two compaction categories, select lower E' value, or average the two values. Percentage Proctor based on laboratory maximum dry density from test standards using 12,500 ft-lb/cu ft (598,000 J/m$^2$) (ASTM D-698, AASHTO T-99, USBR Designation E-11). 1 psi = 6.9 KPa.

**Figure 8-3: Value of E' for Pipe Embedment (PPI 2008)**

| Type of Soil | Depth of Cover, ft | E' for Standard AASHTO Relative Compaction, lb/in$^2$ | | | |
|---|---|---|---|---|---|
| | | 85% | 90% | 95% | 100% |
| Fine-grained soils with less than 25% sand content (CL, ML, CL-ML) | 0-5 | 500 | 700 | 1000 | 1500 |
| | 5-10 | 600 | 1000 | 1400 | 2000 |
| | 10-15 | 700 | 1200 | 1600 | 2300 |
| | 15-20 | 800 | 1300 | 1800 | 2600 |
| Coarse-grained soils with fines (SM, SC) | 0-5 | 600 | 1000 | 1200 | 1900 |
| | 5-10 | 900 | 1400 | 1800 | 2700 |
| | 10-15 | 1000 | 1500 | 2100 | 3200 |
| | 15-20 | 1100 | 1600 | 2400 | 3700 |
| Coarse-grained soils with little or no fines (SP, SW, GP, GW) | 0-5 | 700 | 1000 | 1600 | 2500 |
| | 5-10 | 1000 | 1500 | 2200 | 3300 |
| | 10-15 | 1050 | 1600 | 2400 | 3600 |
| | 15-20 | 1100 | 1700 | 2500 | 3800 |

**Figure 8-4: Values of E' Including Depth (PPI 2008)**

8.3.2 Modulus of Soil Reaction, E'

The modulus of soil reaction is used to measure the influence of the soil in supporting the more flexible pipe. M. G. Spangler (1941) published the Iowa Formula to predict horizontal deflection of flexible pipe. Spangler referred to the soil stiffness as the modulus of passive resistance (e). Watkins and Spangler modified the modulus of passive resistance to the Modulus of Soil Reaction (E') in 1958. However, the modulus of soil reaction was too simplistic since it only had one value for compacted soils. Howard (2006) published a table of E' values that depended on the embedment soil classification that most current standards use. Jeyapalan and Watkins (2004) analyzed multiple pipes around the world using the values determined by Howard (2006) and concluded that the degree of error was unacceptable. The results are summarized in Appendix E. The error is due to the data from field test and variables that were not carefully reviewed. Jeyapalan and Watkins (2004) provided the following list of factors that can affect the soil modulus:

- Native soil type
- Native soil compaction density
- Modulus of native soil
- Trench material type
- Trench material compaction density
- Modulus of trench material
- Size of pipe
- Pipe stiffness - soil stiffness ratio
- Depth of cover
- Trench width – pipe diameter ratio
- Location of water table

Jeyapalan and Watkins (2004) pointed out that "E' is not a fundamental geotechnical engineering property of the soil and cannot be measured either in the laboratory or in the field. It is an empirical soil-pipe system parameter that can only be obtained from back-calculating by knowing the values of the other parameters in the modified Iowa equation."

## 8.4 Terminology

The following pipe terminology will be used throughout this and subsequent sections of the report. A cross-section view showing the different terminology for the area of the pipe is shown in Figure 8-5.

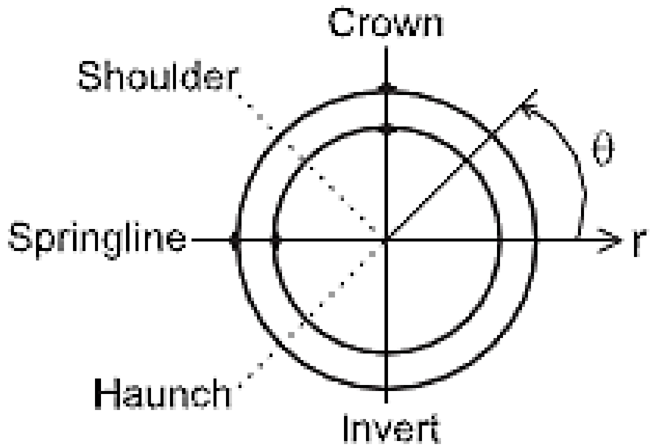



**Figure 8-5: Locations in Pipe Cross-Section (Brachman, Moore, and Rowe 2001)**

Gabriel (2008) gives the terminology of the soil around the pipe shown below and Figure 8-6 illustrates the terminology:

Bedding – Establishes the line and grade for the pipe and gives a firm support.

Foundation – The soil below the bedding that ensures a stable support system. If the native soil at the bottom of the trench is not adequately strong to handle the load then a stronger foreign soil can be placed.

Haunching – The haunch zone is between the springline and bedding. The haunch is a key area of support for the pipe.

Springline – The springline is at the mid-height of the pipe.

Initial Backfill – The initial backfill is the soil above the springline up to at least 12 inches. It provides pipe support and protects the pipe from stones that come from the final backfill.

Final Backfill – The final backfill needs to be strong enough to handle the loads coming from the ground but does not have a significant importance to the pipe.

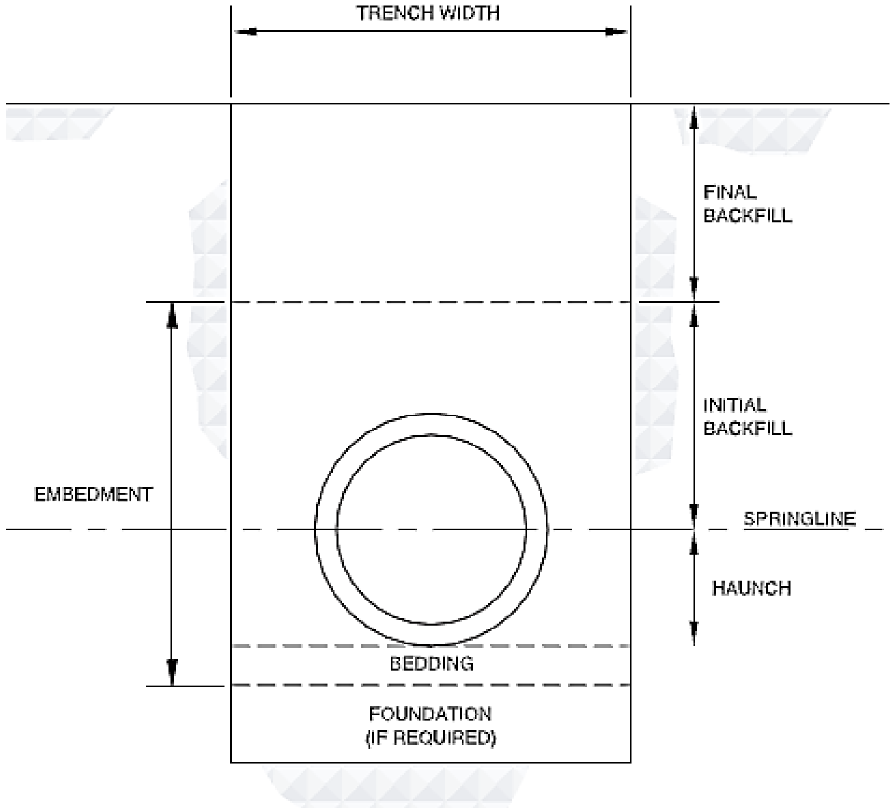



**Figure 8-6: Trench Terminology (PPI 2008)**

## 8.5 Testing Facilities

### 8.5.1 Vector Engineering Testing Facility

The Vector Engineering laboratory in Grass Valley, California houses a soil box test apparatus. The soil box size is made of steel and is 2.5 feet (760 mm) long, 1.97 feet (600 mm) wide and 2.03 feet (620 mm) deep. It is small compared to other soil boxes and therefore it can only handle smaller pipes; a diagram of it is provided in Figure 8-7. The box is loaded with a bladder that can be pressurized up to 39,700 psf (1900 kPa) that simulates a maximum of approximately 100 meters (330 feet) of soil. The bladder style loading creates a uniform load across the top of the structure (Smith et al. 2005). The pipes tested in the simulator are up to 180 mm (7.1 inch), which leaves over 1.5 diameters of space between the edge of the pipe and the edge of the steel box.

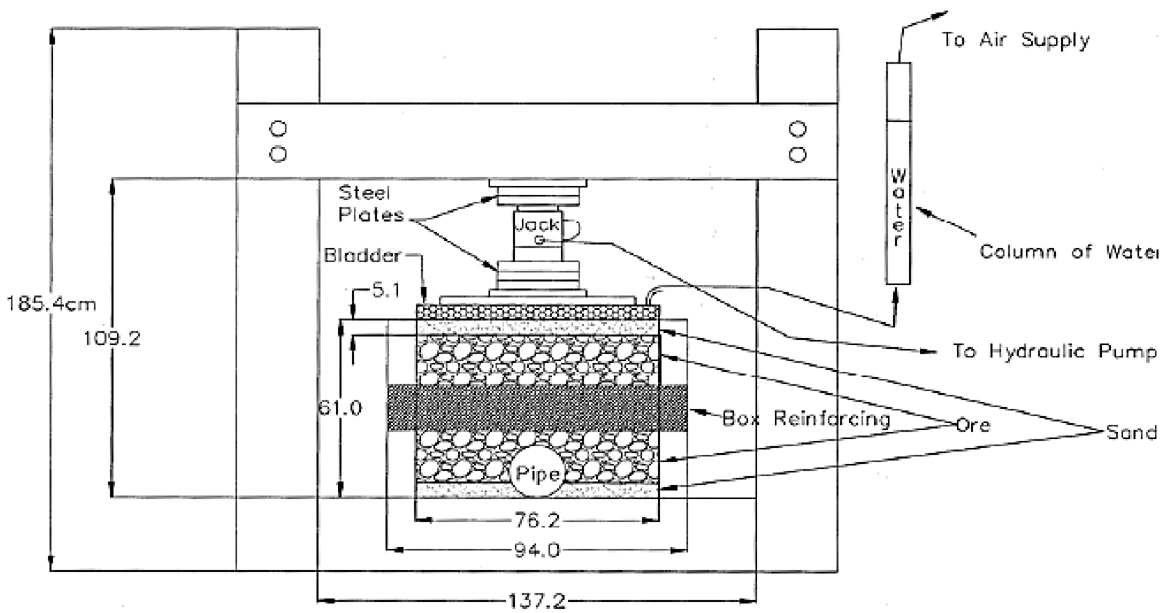



**Figure 8-7: Vector Engineering Testing Apparatus (Smith et al. 2005)**

8.5.2 West Virginia Testing Facility

West Virginia University designed and constructed a steel testing apparatus to test pipe-soil interaction. The dimensions of the testing apparatus are 40 inches long, 25 inches wide, and 20 inches deep (1020 mm long, 640 mm wide, and 510 mm deep). The apparatus was designed to be smaller so it could be transported easily; it can test pipes from 6 to 8 inches in diameter. This leaves at least one pipe diameter of soil below the pipe and on its sides. The loading system is a group of motorcycle inner tubes that are designed for at least 60 psi of pressure. The multiple tubes were placed across the top of the structure so the load would be even across the top. However, it was not intended to surpass 30 psi of pressure. The walls were smooth steel plates that limited the side wall friction. A picture of the testing apparatus is shown in Figure 8-8 (Simmons 2002).

**Figure 8-8: WVU Testing Apparatus (Simmons 2002)**

8.5.3 Utah State

Utah State University has a large scale soil cell made of concrete. The cell uses hydraulic jacks to create a surcharge load at the top of the soil in the test cell. Ten beams span across the soil with five hydraulic jacks each that can place a pressure of 20,000 psf on the soil. The beams are pinned at one end so that they can be rotated to the side. This allows the soil and equipment to access the test cell. The test cell is 24 feet long, 18 feet high, and 15 feet wide and can test pipe up to 60 inches in diameter. An illustration of the test cell is provided in Figure 8-9 (Watkins 1990; Watkins and Anderson 1999).

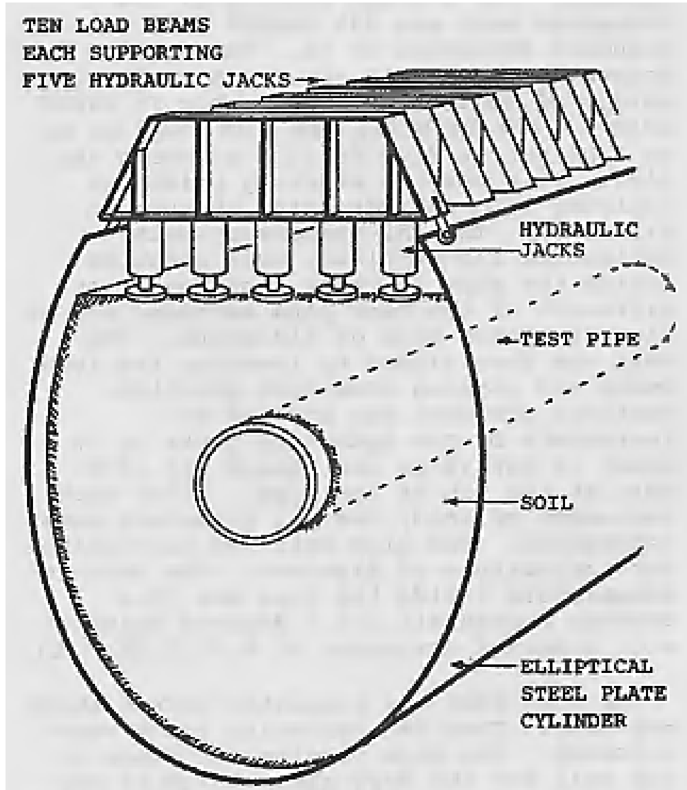



**Figure 8-9: Utah State University Test Frame (Watkins and Anderson 1999)**

8.5.4 Ohio University

Ohio University designed and constructed a load frame where most of the frame is underground. The load frame is structural wide flange beams that are connected to eight ground tension anchors and four concrete columns. The load is applied by two hydraulic cylinders that are 6 square foot and that are held in place by the beams that transfer the load to the columns and tension anchors. Their original soil is used to provide the cell with walls that can be seen in Figure 8-10 (Fernando 2011).

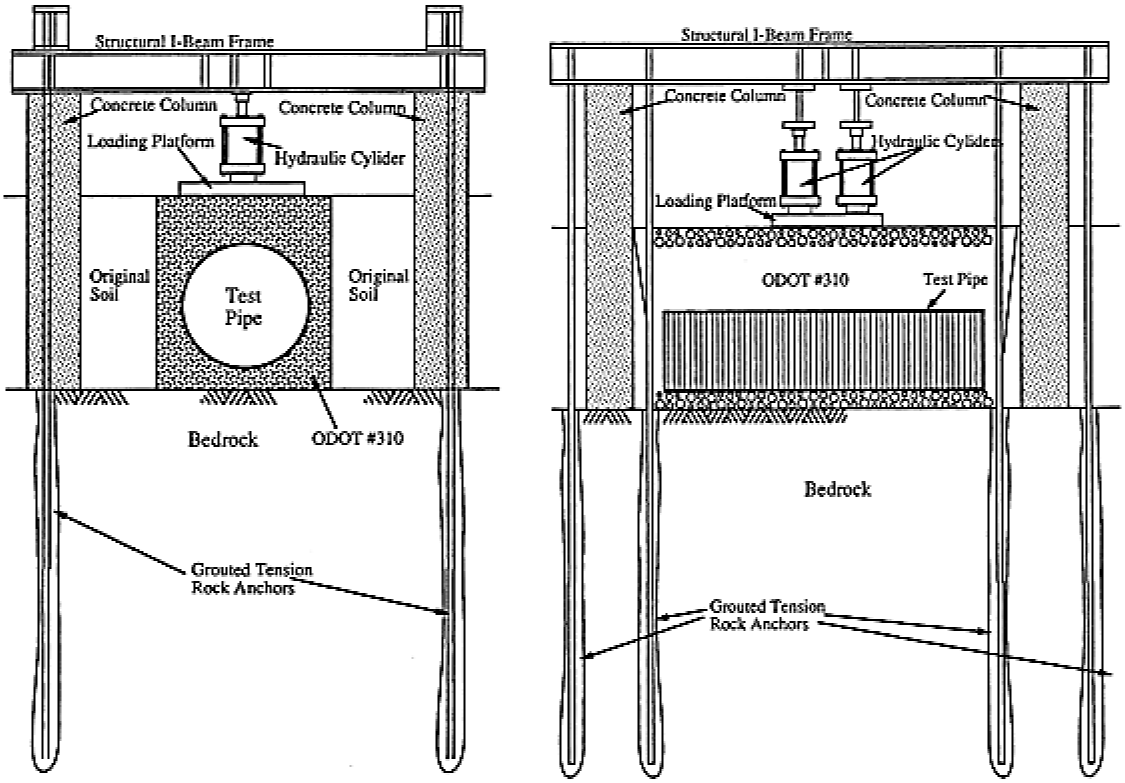



**Figure 8-10: Ohio University Test Cell (Fernando 2011)**

Brachman, Moore, and Rowe (1996) published an interpretation of the Ohio University test cell. They concluded that the hydraulic cylinders created a non-uniform loading since the loading plates are flat, rigid plate where the soil is uneven. Also, the soil is non-homogenous so it will not evenly consolidate while being loaded that will lead to stress concentrations in the pipe. The uneven stress concentration creates a situation that is different than what the pipe would feel under deep burial (Brachman, Moore, and Rowe 1996).

8.5.5 Western Ontario Testing Facility

The University of Western Ontario designed a steel testing facility to test plastic pipe. The dimensions of the test cell are 6.56 feet (2000 mm) in both directions and 5.25 feet (1600 mm) tall. The structure is made to simulate up to 145 psi (1000 kPa) of vertical pressure that amounts to 174 feet of 120 pcf soil (53 meters of 19 $kN*m^3$ soil) with a pressurized bladder made of 3 mm thick Buna N rubber (Brachman, Moore, and Rowe 2000). The side walls have two 0.1 mm polyethylene sheets that are lubricated with silicone grease (Dow Corning 44 high-temperature bearing grease) to limit the friction to roughly 5%. Western Ontario concluded that the sidewall friction error is minimized with a friction angle below 5% (Brachman, Moore, and Rowe 2001). The testing apparatus is depicted in Figure 8-11 and Figure 8-12.

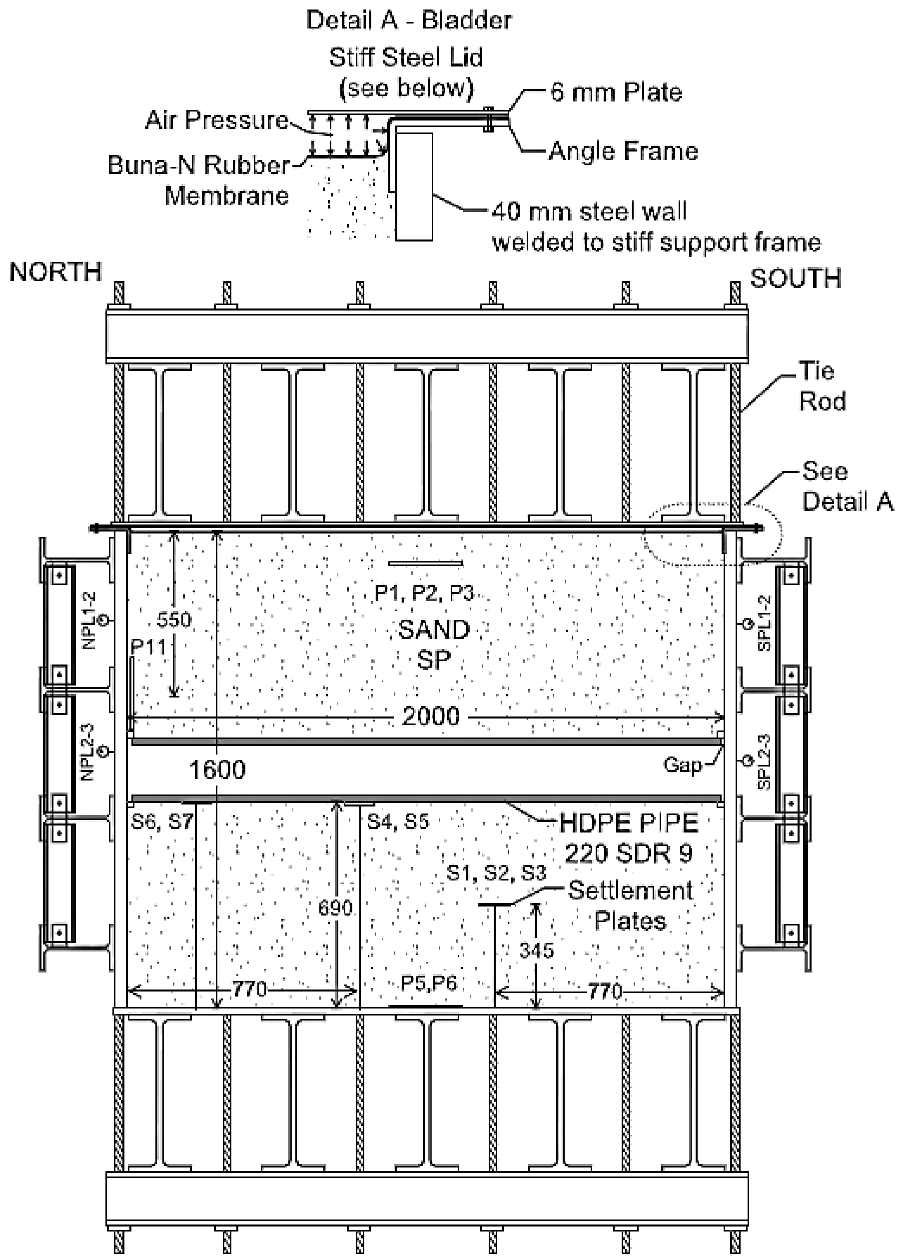



**Figure 8-11: Longitudinal Section of Western Ontario's Testing Apparatus (Brachman, Moore, and Rowe 2001)**

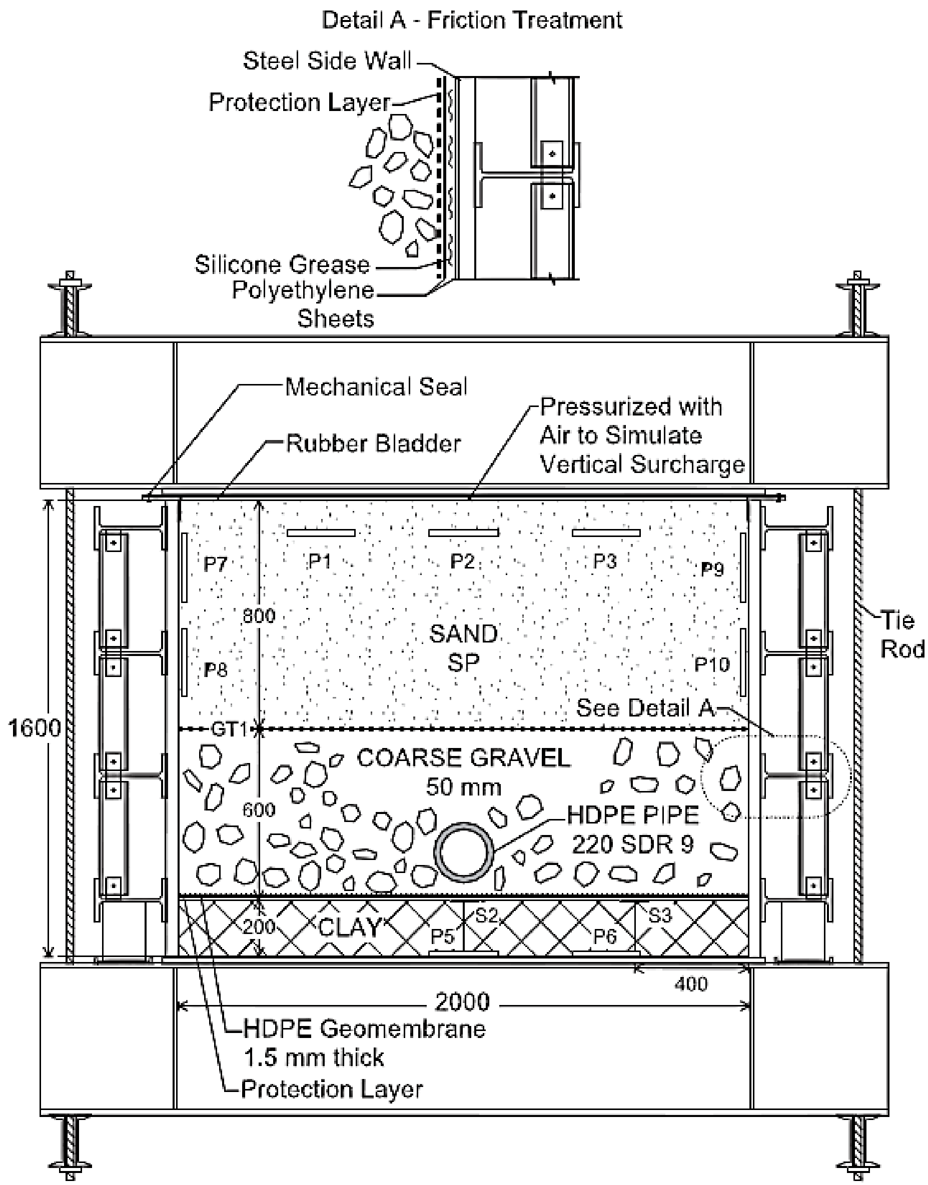



**Figure 8-12: Transverse Section of Western Ontario's Testing Apparatus (Brachman, Moore, and Rowe 2001)**

## 8.6 Joint Testing

The joints in buried pipe can be characterized as either moment release joints (those designed to accommodate rotation that reduces the longitudinal bending moments close to zero) or moment transfer joints (those that limit rotation and transfer longitudinal bending moments from one pipe to the next) (Garcia 2012). An example of a moment transfer joint is a band connection; an example of a moment release is a gasketted bell and spigot joint, which is common for thermoplastic pipes due to their fast and easy assembly and water tightness. The moment release joints must accommodate the vertical shear force and rotation acting at the joint (Moore et al. 2012). Specifically for PVC joints, the main type of gasketted bell and spigot joint is the Reiber joint, which is reinforced with an adjoining external and internal ring that secures the gasket. (Rahman and Bird 2006).

ASTM D3212 defines a test for the quality control of pipe joints, and discusses joints for polymer pipes with both pressurized and vacuum test within the specification.

Buco et al. (2008) developed an experimental apparatus to test concrete joints, as illustrated in Figure 8-13. The apparatus has the capability to test longitudinal bending, axial, and vertical shear by changing the boundary conditions and force from the load cell. Figure 8-13 shows the configuration for the longitudinal bending test. For the vertical shear test the load cell is moved to point B and support 2 is fixed. However, this test does evaluate the strength of the joint in a pipe-soil system.

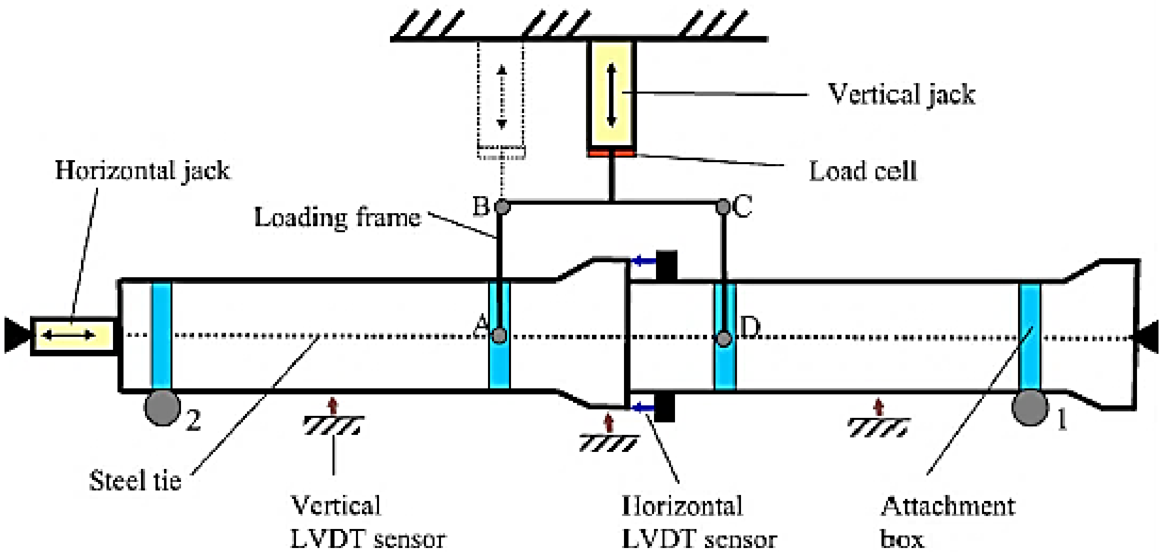



**Figure 8-13: Queen's University Joint Testing Apparatus (Buco et al. 2008)**

## 8.7 Incorrect Installation

### 8.7.1 Incorrect Joint Installation

Since flexible pipes get much of their strength from the soil interaction in pipe-soil systems, the installation of the pipe becomes integral. Many studies assume that the soil is installed correctly without deficiencies; however, in real world construction this may not be true. Jeyapalan and Abdel-Magid (1987) studied failures of reinforced polymer pipes with different non-uniform bedding conditions using finite element analysis. The different types of joint construction errors are illustrated in Figure 8-14. Buco et al. (2006) used stochastic analysis to study nonreinforced concrete sewers and specific nonuniform bedding condition were analyzed. The different bedding conditions are shown in Figure 8-15. The the nonuniform bedding conditions can come from improper installations or due to leaks in the pipe causing erosion around the leak.

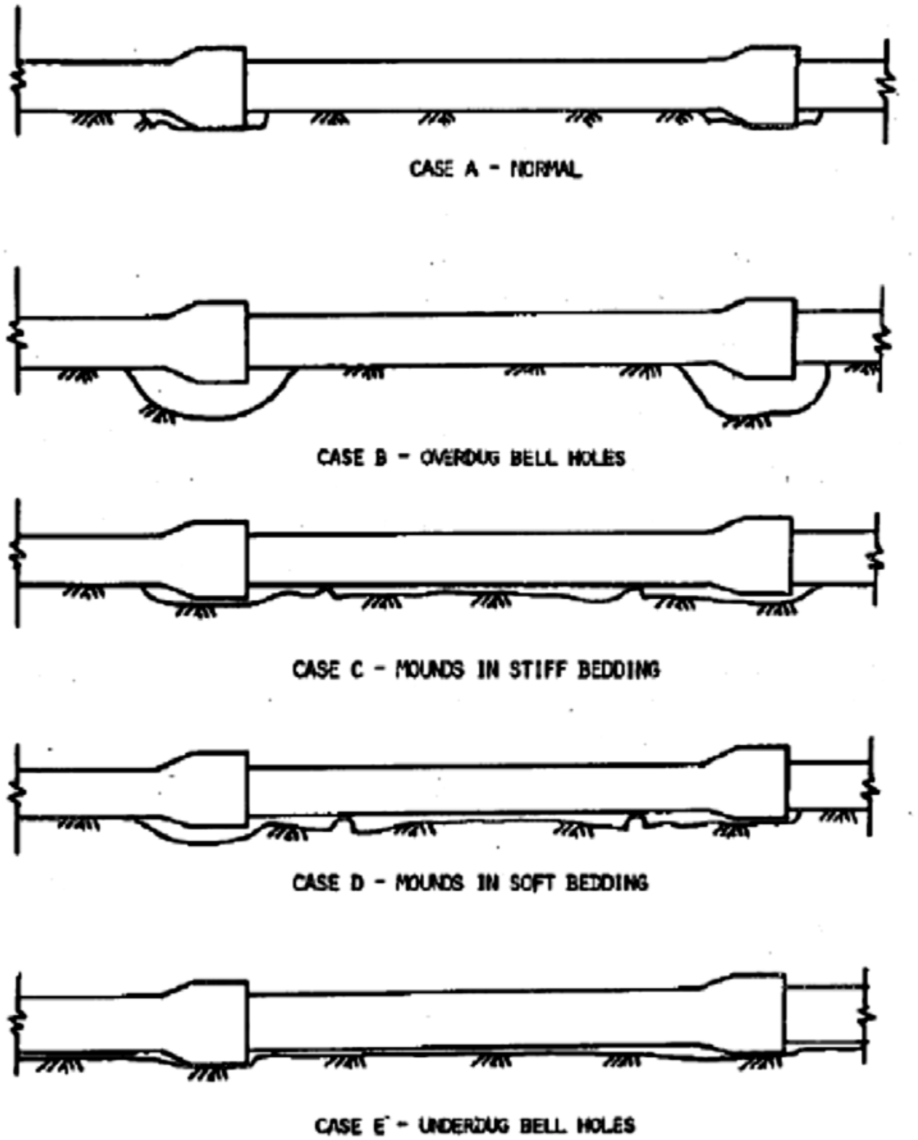
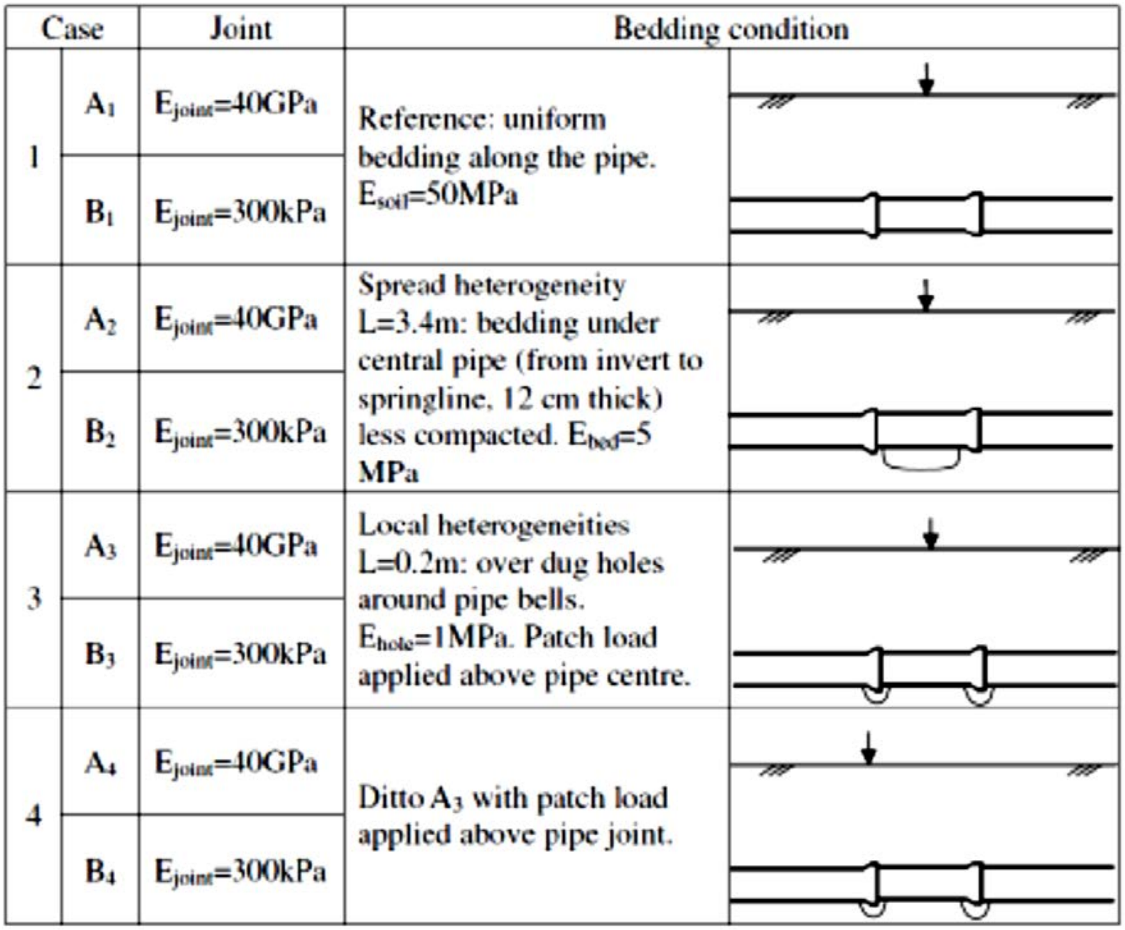



**Figure 8-14: Joint Construction Errors (Jeyapalan and Abdel-Magid 1987)**



| Case | | Joint | Bedding condition | |
|---|---|---|---|---|
| 1 | $A_1$ | $E_{joint}$=40GPa | Reference: uniform bedding along the pipe. $E_{soil}$=50MPa | |
| | $B_1$ | $E_{joint}$=300kPa | | |
| 2 | $A_2$ | $E_{joint}$=40GPa | Spread heterogeneity L=3.4m: bedding under central pipe (from invert to springline, 12 cm thick) less compacted. $E_{bed}$=5 MPa | |
| | $B_2$ | $E_{joint}$=300kPa | | |
| 3 | $A_3$ | $E_{joint}$=40GPa | Local heterogeneities L=0.2m: over dug holes around pipe bells. $E_{hole}$=1MPa. Patch load applied above pipe centre. | |
| | $B_3$ | $E_{joint}$=300kPa | | |
| 4 | $A_4$ | $E_{joint}$=40GPa | Ditto $A_3$ with patch load applied above pipe joint. | |
| | $B_4$ | $E_{joint}$=300kPa | | |

**Figure 8-15: Bedding Conditions (Buco et al. 2006)**

8.7.2 Incorrect Pipe Installation

Incorrect installation along the pipe can be improper bedding (Figure 8-16) or improper support conditions (Figure 8-17). Specifically, the problem of inadequate haunch zone compaction is important. Zoladz et al. (1996) concluded that rammer compacting and tamping the soil in the haunch zone was the most effective compared to vibratory plates and shovel slicing.

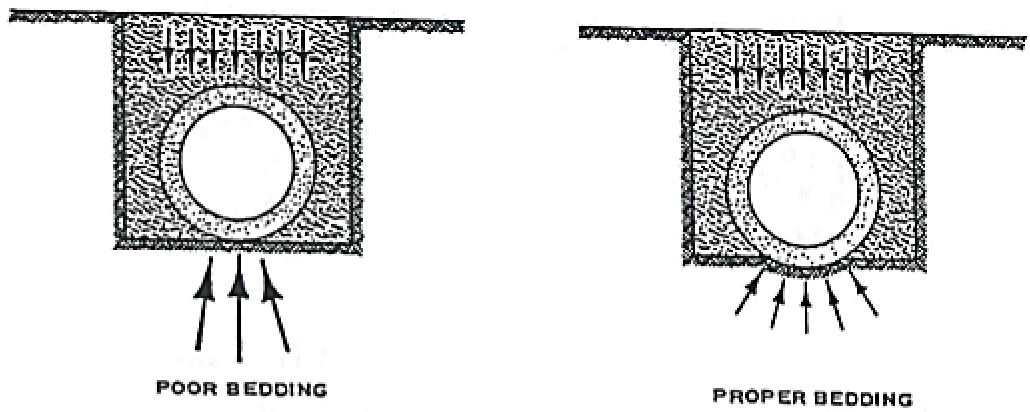



**Figure 8-16: Effect of Bedding (Allen 2013)**

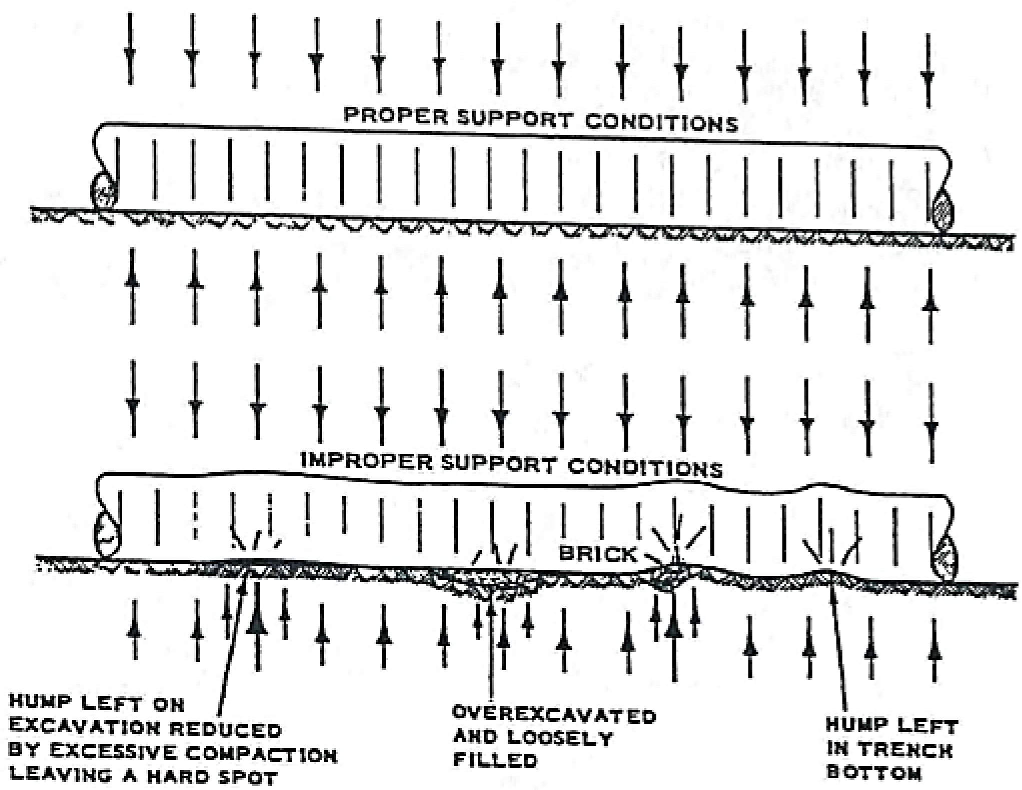



**Figure 8-17: Proper and Improper Support Conditions (Allen 2013)**

Whenever rock is used to support the pipe, there needs to be adequate earth cushion between the pipe and rock so that the load can be distributed. This problem is illustrated in Figure 8-18.

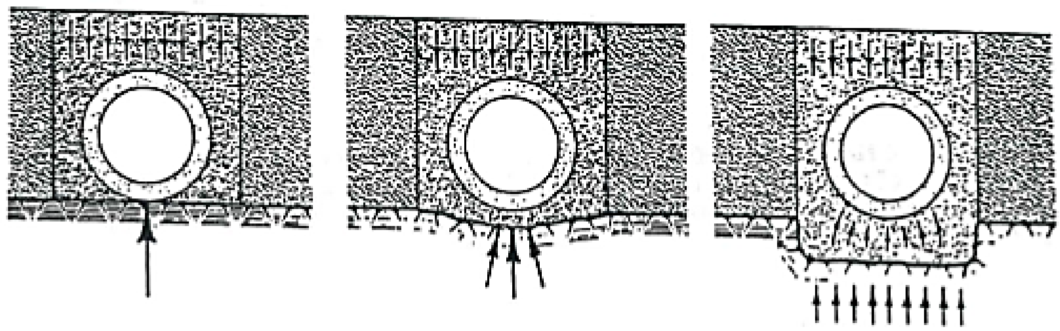


**Figure 8-18: Pipe Installed on Rocks (Allen 2013)**

## 8.8 Live Loading and Minimum Cover Requirements

### 8.8.1 ASTM and AASHTO Minimum Cover Requirements

ASTM D2321 allows the engineer to recommend a soil cover, but if there is no guidance from an engineer then ASTM requires at least 24 inches (0.6 m) of cover or one pipe diameter (whichever is larger) for Class 1A and 1B embedment, and at least 36 inches (0.9 m) or one pipe diameter for Class II, III, and IVA embedment. The cover is measured from the top of the pipe to the bottom of the pavement for flexible pavement and from the top of the pipe to the top of the pavement for rigid pavement (PPI 2008).

AASHTO recommends at least one foot (0.3 m) of cover over the pipe crown for 4-inch to 48-inch (0.1 m to 1.2 m) diameter pipe for a conservative limit as shown in Table 8-1. The backfill envelope should provide a minimum E' value of 1,000 psi (6,900 kPa). These numbers were found using a Class III material compacted to 90% Standard Proctor Density, although other material can provide similar strength at slightly lower levels of compaction. Structural backfill material should extend 6 inches (0.15 m) over the crown of the pipe; the remaining cover should be appropriate for the installation (ADS 2014).

**Table 8-1: Minimum Cover Requirements per AASHTO (ADS 2014)**

| Inside Diameter, ID, in.(mm) | Minimum Cover ft. (m) | Inside Diameter, ID, in.(mm) | Minimum Cover ft. (m) |
|---|---|---|---|
| 4 (100) | 1 (0.3) | 24 (600) | 1 (0.3) |
| 6 (150) | 1 (0.3) | 30 (750) | 1 (0.3) |
| 8 (200) | 1 (0.3) | 36 (900) | 1 (0.3) |
| 10 (250) | 1 (0.3) | 42 (1050) | 1 (0.3) |
| 12 (300) | 1 (0.3) | 48 (1200) | 1 (0.3) |
| 15 (375) | 1 (0.3) | 54 (1350) | 2 (0.6) |
| 18 (450) | 1 (0.3) | 60 (1500) | 2 (0.6) |

8.8.2 Live Load and Cover Testing

Multiple research groups have conducted tests on flexible HDPE and PVC pipe buried under shallow cover. The recommended minimum amount of cover allowed is 12 inch (305 mm) based upon the numerical model developed by Klaiber et al. (1996).

Trickey and Moore (2007) used finite-element modeling to evaluate soil cover and found that the deflection of the pipe decreased significantly as the soil cover increased. Field test were performed in Florida where the cover was divided into three test groups, 0.5D, 1D, and 2D (D=diameter), with the minimum cover being 1.5 feet. A HS-20 truck was used for the loads by passing over the pipes 1000 times. It was found that the pressure decreased significantly from the 0.5D to 1D and an insignificant amount of pressure decreased from 1D to 2D. The test showed that if there is at least 3 feet of cover, the load on the pipe becomes very small. The maximum load on the pipes with 0.5D cover was 71.3 psi (490 kPa), which resulted in a deflection of 0.21 inches (5 mm) vertical and 0.11 inches (2.79 mm) horizontal. The maximum load on the pipes with 1D cover was 10.85 psi (75 kPa), which resulted in a deflection of 0.07 inches (1.78 mm) vertical and 0.088 inches (2.24 mm) horizontal. The maximum load on the pipes with 2D cover was 2.9 psi (20 kPa), which resulted in a deflection of 0.065 inches (1.65 mm) vertical and 0.021 inches (0.53 mm) horizontal. The pipe buried with only 0.5D of cover might have had slippage problems (Chaallal et al. 2014). The deformation and strain in the pipe increased less after every cycle because the pipe-soil system became more stable due to compaction from the repeated load (Chaallal et al. 2014; Faragher et al. 2000).

Stuart et al. (2011) used the finite-element software CANDE-2007 to conclude that the minimum cover that many DOT's use was not conservative when looking at long-term performance of the pipe. Minimum cover requirements were found with different pavement condition (rigid, flexible, and no pavement) and different soils (gravely sand, silty clay, and silty sand). More cover was required for flexible pavement than rigid pavement as expected, and when there was no pavement the controlling situation was during construction since the cover soil is usually not compacted during construction. The flexible pavement was asphalt concrete and the rigid pavement was Portland cement concrete. The most minimum cover required was for silty clay compacted to 80% with six inch thick asphalt concrete with the HDPE needing 7.5 feet (2.3 m) and the PVC needing 7.8 feet (2.4 m). The scenario with the least minimum cover required was gravely sand compacted to 90% and one foot thick Portland cement concrete with the required cover of 1.8 feet (0.6 m) for PVC and 1.5 feet (0.5 m) for HDPE. The worst case scenario for no pavement was during construction due to less soil compaction with the most minimum soil cover required was for with silty clay compacted to 80% with 7.0 feet (2.1 m) for HDPE and 5.2 feet (1.6 m). The results of the no pavement soil cover required matched well with the AASHTO requirements.

## 8.9 Long Term and Temperature

8.9.1 Vector Engineering Long Term Temperature Calculation

Vector Engineering conducted tests on pipes at four different temperatures of 22, 40, 50, and 60 $^{o}$C (72, 104, 122, and 140 $^{o}$F) and compared the results to the Burns-Richard model (BR Model). The gravel and pipes were heated with a heat gun days before the loads were placed (Smith et al. 2005). The increase in temperature from 23 to 50 $^{o}$C caused a decrease in the modulus of elasticity of 50%. This is true under the assumption that the BR model is accurate under extreme depths, which Smith believes to be accurate. Tables 8-2 and 8-3 present some of the results (Smith 2004).

**Table 8-2: Laboratory vs Calculated Deflections – 160 mm Dual Wall Pipe (Smith 2004)**

| Heap | Vertical Deflection, % | | |
|---|---|---|---|
| Height | Laboratory | Burns-Richard | Burns-Richard |
| m | 23°C, short-term | 23°C, short-term | 50°C, long-term |
| 20 | 4.3 | 5.0 | 8.0 |
| 40 | 7.2 | 8.6 | 13.3 |
| 60 | 11.6 | 11.6 | 17.6 |
| 80 | 13.8 | 14.3 | 21.5 |
| 100 | 15.2 | 16.8 | 24.9 |
| 120 | 19.2 | 19.2 | 28.1 |
| 140 | ND | 21.4 | 31.0 |

Note: Gravel compacted to 88% standard Proctor. Soil modulus calibrated at 120 m.

**Table 8-3: Polyethylene Modulus of Elasticity (Smith 2004)**

| Temperature °C | Modulus of Elasticity, mPa | |
|---|---|---|
| | Short-Term | Long-Term |
| 23 | 990 | 240 |
| 30 | 860 | 200 |
| 45 | 640 | 120 |
| 50 | 510 | 110 |

8.9.2 Western Ontario Long Term Testing

Krushelnitzky and Brachman (2013) tested the long term performance at different temperatures using the Western Ontario soil box. The test was for a solid waste facility where the ground temperatures can get higher than normal.

The fluid pressure from the rubber membrane reached 72.5 psi (500 kPa) for each test that simulates 98 to 131 feet (30 to 40 m) of solid waste depth. The vertical pressure was applied in increments. First, a pressure of 29 psi (200 kPa) was rapidly applied and then held constant for 20 minutes. Next, the pressure was rapidly increased to 58 psi (400 kPa) and also then held for 20 minutes. Last, the pressure was increased to 73 psi (500 kPa) and then held constant for 1000 hours (41 days) (Krushelnitzky and Brachman 2013).

The temperature was applied through an air heating system that circulated hot air through the pipe. The pipes were heated to temperatures of 72, 122, and 176 $^{o}$F (22, 50, and 80 $^{o}$C, respectively) and kept within ±1 $^{o}$F (±0.5 $^{o}$C). The pipes were heated 24 hours before each test so that no significant stresses would develop from the pipe wanting to expand due the heat. The gradient for the pipes was less than 2 $^{o}$F (1 $^{o}$C). Heating to 50 and 176 $^{o}$F (50 and 80 $^{o}$C) was found to lead to diameter increases of 0.02 and 0.05 inches (0.5 and 1.2 mm) when the pipe was constrained and length increases of 0.08 and 0.2 inches (2 and 5 mm) when the pipe was unconstrained (Krushelnitzky and Brachman 2013).

All pipes tested were 4 inch (100 mm) nominal diameter; the only change between the pipes was the temperature.  The ends of the pipe were not restrained so no axial compression forces would develop from the thermal expansion.  The pipes were the same type and manufacturer to limit any differences.  The changes in deflection were measured using potentiometers (Celesco MTA-3E-5KC draw wire displacement transducers), which were placed at the center of the cell where the boundary conditions would have the smallest effect on the results (Krushelnitzky and Brachman 2013).

The vertical deflections were measured after 20 minutes under 73 psi (500 kPa) of pressure.  The vertical deflections were -1.9, -2.3, and -2.5% at 72, 122, and 176 $^{o}$F (22, 50, and 80 $^{o}$C), respectively.  The 176 $^{o}$F (80 $^{o}$C) vertical deflection was 1.2 times greater than the 122 $^{o}$F (50 $^{o}$C) vertical deflection and 1.3 times compared to the 72 $^{o}$F (22 $^{o}$C) vertical deflection.  The horizontal deflections were 1.3, 1.3, and 1.1% at 72, 122, and 176 $^{o}$F (22, 50, and 80 $^{o}$C), respectively.  The horizontal deflection actually decreases as the temperature increased by a factor of 0.85 from 22 to 80 $^{o}$C (Krushelnitzky and Brachman 2013).

The test compared the initial 20 minute deflection to the 1000 hour deflection to see how much of the deflection over time.  After the 1000 hours, the results from the test showed that the pipe deflected 1.4 times the initial vertical and 1.4 times the initial horizontal for the 72 $^{o}$F (22 $^{o}$C); 1.2 times the initial vertical and 1.2 times the initial horizontal for the 122 $^{o}$F (50 $^{o}$C), and 1.2 times the initial vertical and 1.1 times the initial horizontal for the 176 $^{o}$F (80 $^{o}$C). It was concluded that the long term deflection was insignificantly effected by the temperature differences (Krushelnitzky and Brachman 2013).

The test found that most of the deflection occurred in the first 100 hours for all three temperatures. The deflections after the initial 100 hours stabilized but the pipes continued to deflect with time. Two different methods were used to predict the long term deflection past 1000 hours (Krushelnitzky and Brachman 2013). The first method was by using the Chua and Lytton (1989) analytical model, which was only accurate for the 72 $^{o}$F (22 $^{o}$C) pipe when compared to the 1000 hour results. Using the model to extrapolate to a 50 year deflection, the model found that the pipe would vertically deflect -3.4% at 72 $^{o}$F (22 $^{o}$C), which is 1.3 times more than the 1000 hour vertical deflection and determined that the 50 year horizontal deflection would be 2.9% at 72 $^{o}$F (22 $^{o}$C) that is 1.5 times the 1000 hour horizontal deflection. The other method was to extrapolate the results to 50 year deflection by using the slope from 100 to 1000 hours log cycle. The vertical diameter change was predicted to be -3.2% using this method for all three pipe temperatures. Both methods predicted that the deflection would be less than 5% (Krushelnitzky and Brachman 2013).

## 8.10 Maximum Cover Requirements

### 8.10.1 ASTM Maximum Cover Requirements

The maximum burial depth for corrugated HDPE pipes is shown in Table 8-4. The results are for pipes installed in accordance with ASTM D2321 and assume zero hydrostatic load and that the native soil is of adequate strength and suitable for installation. (ADS 2015)

**Table 8-4: Maximum Cover Requirements for Corrugated HDPE Pipe per ASTM F2648 (ADS 2015)**

| Diameter in. (mm) | Class One | | Class Two | | | | Class Three | |
|---|---|---|---|---|---|---|---|---|
| | Compacted | | 95% | | 90% | | 95% | |
| 4 (100) | 34 | (10.4) | 23 | (7.0) | 16 | (4.9) | 17 | (5.2) |
| 6 (150) | 40 | (12.2) | 27 | (8.2) | 19 | (5.8) | 19 | (5.8) |
| 8 (200) | 30 | (9.1) | 21 | (6.4) | 14 | (4.3) | 15 | (4.6) |
| 10 (250) | 34 | (10.4) | 23 | (7.0) | 16 | (4.9) | 17 | (5.2) |
| 12 (300) | 35 | (10.7) | 24 | (7.3) | 17 | (5.2) | 18 | (5.5) |
| 15 (375) | 37 | (11.3) | 25 | (7.6) | 18 | (5.5) | 18 | (5.5) |
| 18 (450) | 32 | (9.8) | 22 | (6.7) | 15 | (4.6) | 16 | (4.9) |
| 24 (600) | 27 | (8.2) | 19 | (5.8) | 13 | (4.0) | 13 | (4.0) |
| 30 (750) | 22 | (6.7) | 16 | (4.9) | 11 | (3.4) | 11 | (3.4) |
| 36 (900) | 26 | (7.9) | 18 | (5.5) | 12 | (3.7) | 13 | (4.0) |
| 42 (1050) | 24 | (7.3) | 17 | (5.2) | 11 | (3.4) | 12 | (3.7) |
| 48 (1200) | 23 | (7.0) | 16 | (4.9) | 11 | (3.4) | 11 | (3.4) |
| 60 (1500) | 26 | (7.9) | 18 | (5.5) | 12 | (3.7) | 12 | (3.7) |

### 8.10.2 AASHTO Maximum Cover Requirements

The maximum burial depth for corrugated HDPE pipes according to AASHTO is shown in Table 8-5. The results are for pipes installed in accordance with ASTM D2321 and assume zero hydrostatic load, incorporate the maximum safety factors represented section of the *Drainage Handbook*, and that the native soil is of adequate strength and is suitable for installation. (ADS 2014)

**Table 8-5: Maximum Cover Requirements for Corrugated HDPE Pipe per AASHTO (ADS 2015)**

| Diameter | Class One | | | | Class Two | | | | | | Class Three | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| in. (mm) | Compacted | | Dumped | | 95% | | 90% | | 85% [3] | | 95% | | 90% [3] | |
| 4 (100) | 37 | (11.3) | 18 | (5.5) | 25 | (7.6) | 18 | (5.5) | 12 | (3.7) | 18 | (5.5) | 13 | (4.0) |
| 6 (150) | 44 | (13.4) | 20 | (6.1) | 29 | (8.8) | 20 | (6.1) | 14 | (4.3) | 21 | (6.4) | 15 | (4.6) |
| 8 (200) | 32 | (9.8) | 15 | (4.6) | 22 | (6.7) | 15 | (4.6) | 10 | (3.0) | 16 | (4.9) | 11 | (3.4) |
| 10 (250) | 38 | (11.6) | 18 | (5.5) | 26 | (7.9) | 18 | (5.5) | 12 | (3.7) | 18 | (5.5) | 13 | (4.0) |
| 12 (300) | 38 | (11.6) | 18 | (5.5) | 26 | (7.9) | 18 | (5.5) | 13 | (4.0) | 19 | (5.8) | 14 | (4.3) |
| 15 (375) | 42 | (12.8) | 20 | (6.1) | 28 | (8.5) | 20 | (6.1) | 14 | (4.3) | 20 | (6.1) | 15 | (4.6) |
| 18 (450) | 35 | (10.7) | 17 | (5.2) | 24 | (7.3) | 17 | (5.2) | 12 | (3.7) | 17 | (5.2) | 12 | (3.7) |
| 24 (600) | 30 | (9.1) | 15 | (4.6) | 21 | (6.4) | 15 | (4.6) | 10 | (3.0) | 15 | (4.6) | 11 | (3.4) |
| 30 (750) | 25 | (7.6) | 12 | (3.7) | 18 | (5.5) | 12 | (3.7) | 8 | (2.4) | 13 | (4.0) | 9 | (2.7) |
| 36 (900) | 29 | (8.8) | 13 | (4.0) | 20 | (6.1) | 13 | (4.0) | 9 | (2.7) | 14 | (4.3) | 9 | (2.7) |
| 42 (1050) | 27 | (8.2) | 13 | (4.0) | 19 | (5.8) | 13 | (4.0) | 8 | (2.4) | 13 | (4.0) | 9 | (2.7) |
| 48 (1200) | 25 | (7.6) | 12 | (3.7) | 17 | (5.2) | 12 | (3.7) | 7 | (2.1) | 12 | (3.7) | 8 | (2.4) |
| 54 (1350) | 26 | (7.9) | 12 | (3.7) | 18 | (5.5) | 12 | (3.7) | 7 | (2.1) | 12 | (3.7) | 8 | (2.4) |
| 60 (1500) | 29 | (8.8) | 13 | (4.0) | 20 | (6.1) | 13 | (4.0) | 8 | (2.4) | 14 | (4.3) | 9 | (2.7) |

## 8.11 Pipe Testing

### 8.11.1 Parallel Plate Test

The parallel plate test, ASTM D2412 - *Standard Test Method for Determination of External Loading Characteristics of Plastic Pipe by Parallel-Plate Loading*, is used to measure the pipe stiffness, stiffness factor, and the load at certain locations (ASTM D2412). "A short length of pipe is loaded between two rigid flat plates at a controlled rate of approach to one and another. Load-deflection (of the pipe diameter) data obtained. If cracking, crazing, delamination, or rupture occurs then the corresponding load and deflection are recorded." Thermoplastic pipe shall be 6 plus or minus 1/8 inch long. AASHTO M294 - *Corrugated Polyethylene Pipe, 300- to 1500-mm Diameter* and AASHTO M304 specify ASTM D2412 for the analysis of the pipe stiffness with some slight modification including how many pipes needed to be tested. ASTM D2412 shows how to calculate the pipe stiffness with the equation:

$$PS = \frac{F}{\Delta y} \text{ (lbf/inch/inch)} \qquad \text{(Equation 8-1)}$$

Where $PS$ is the pipe stiffness obtained by dividing the force per unit length of specimen by the resulting deflection in the same units at the prescribed percentage deflection, $F$ is the load applied to the pipe to produce a given percentage deflection, pounds-force per linear inch, and $\Delta y$ is the measured change of the inside diameter in the direction of load application, inches. However, this does not give accurate results for the pipe stiffness within the soil. Gabriel (2008) concludes that the parallel plate test, "is not representative of a typical installation and is not accurate for predicting field performance."

### 8.11.2 Pipe Flattening

AASHTO M304 and M294 both require a flattening test to measure the ability of HDPE and PVC pipe to deform but not fail. The specimen is loaded within the parallel plate test the same way as described above. However, as the load is increased the pipe begins to flatten.

AASHTO M304 flattens PVC pipes to 60 percent of its original dimension for pipes less than 318 kPa. If the PVC pipe is above 318 kPa, the pipe must be flattened according to Equation 8-2 as a percentage.

$$\frac{3.43\ OD}{(OD - ID)} \qquad \text{(Equation 8-2)}$$

Where OD and ID are the measured outside and inside diameters, respectively. The rate of loading is uniform within 120 to 300 seconds. Small tears at the end of the beam do not count as failure.

AASHTO M294 recommends flattening the HDPE pipe down 20% of its vertical inside diameter. The specimens are defined as failed if any cracking, splitting, or delamination can be seen.

8.11.3 Brittleness and Impact Resistance

AASHTO M294 tests HDPE pipe specimens for their brittleness. It refers to ASTM D2444 – *Impact Resistance of Plastic Pipe and Fittings* (2002) for the procedure. Six impacts are made on the specimen separated by 120 ± 10 degrees using a Tup B with a mass of 4.5 kg. The tup is dropped 3.0 m and the specimen must be at -4 ± 2 $^{o}$C (24.8 ± 4 $^{o}$F) for 24 hours and the specimen can only be outside of that environment for 60 seconds to conduct the tests. The center of the tup must strike a corrugation crown on every impact (AASHTO M294).

AASHTO M304 (2003) tests impact resistance of PVC pipe according to ASTM D2444. A tup B and flat plate holder is used for the experiment; the test temperature is 23 ± 2 $^{o}$C (73.4 ± 4 $^{o}$F). Six specimens are tested and if one fails then six more specimens must be tested. If 11 out of 12 pass then the product is deemed acceptable. For 100 and 150 mm pipe specimens, a 10 kg Tup B is used; for larger pipes either a 10 kg or 15 kg tup is used. The tup will strike between the ribs. If there is not enough space between the ribs for the tup to make a clean hit, then the center of the tup will strike the rib. The specimen fails if any crack, split, shattering, or separation on ribs, body, or seam occurs (AASHTO M304).

### 8.11.4 Environmental Stress Cracking

AASHTO M294 tests the environmental stress cracking of HDPE pipe and refers to ASTM D1693 - *Standard Test Method for Environmental Stress-Cracking of Ethylene Plastics* for the procedure. D1693 summarizes the procedure as, "Bent specimens of plastic, each having a controlled imperfection on one surface, are exposed to the action of a surface-active agent. The proportion of the total number of specimens that crack in a given time is observed." The test puts the specimens under high multiaxial stresses because of an imperfection in the specimen that typically causes environmental stress cracking. The procedure puts a hole in the pipe specimen and then bends the specimen. The bent specimen is placed in a holder and put into a reagent bath (ASTM D1693).

AASHTO M294 modifies the procedure for HDPE pipe. Three 90-degree arc length specimens are tested. The specimens are reduced to 80% of their arc chord dimension and then placed in bath of Igepal CO-630 (nonylphenoxy polyethyleneoxy) ethanol at $50 \pm 2\ ^{o}C$ ($122 \pm 4\ ^{o}F$) for 24 hours and there should be no cracking of the specimen (AASHTO M294). Hsuan and McGrath (2009) stated that there are three problems with this procedure:

- The failure time of an individual test specimen cannot be recorded.
- There is a large standard deviation.
- The actual stress condition varies throughout the test and is not known, because of stress relaxation in the material.

8.11.5 Slow Crack Growth

AASHTO M294 tests slow crack growth resistance of resin compounds and refers to ASTM F2136 - *Standard Test Method for Notched, Constant Ligament-Stress (NCLS) Test to Determine Slow-Crack-Growth Resistance of HDPE Resins or HDPE Corrugated Pipe* for the procedure. The test method determines the susceptibility of HDPE or corrugated pipe to slow crack growth while the specimen is under ligament stress in an accelerated environment. Dumbbell sections of the HDPE are created with a notch in the middle section. The samples are placed in a bath of 10% of Igepal CO-630 (nonylphenoxy polyethyleneoxy) and 90% deionized water at a temperature of 122 ± 2 $^{o}$F (50 ± 1$^{o}$C). The sample is placed under a constant ligament stress until brittle failure occurs (ASTM F2136). M294 requires the load to be 4100 kPa (600 psi) and the notch depth should be 20% of the nominal thickness (AASHTO M294).

8.11.6 Joint Integrity

AASHTO M294 tests the joint integrity of HDPE pipes using the parallel plate test. The joint is assembled according to the manufacturer's recommendations with two pipe specimens that are at least 300 mm long each. The joint is centered between the parallel plates and a load of 12.5 mm per minute is placed on the joint till 20% deflection of the nominal vertical diameter is reached (AASHTO M294).

AASHTO M304 tests the pipe joints for both soil-tightness and water-tightness. For soil-tightness, the joint is assembled according to the manufacturer's guidelines and the largest joint opening between the pipes and jointing device. The joint must be less than 25 mm wide. For water-tightness, M304 uses ASTM D3212 and 75 kPa (11 psi) water pressure.

## 8.12 Instrumentation

### 8.12.1 Earth Pressure Cell

The earth pressure within the testing apparatus and specifically near the pipe will be used to find the pressure in various areas. The earth pressure cell consists of two circular steel plates that are welded together. Within those steel plates is an area filled with de-aired oil and as the plate push together the fluid pressure inside increases. A vibrating wire pressure transducer reads the pressure by converting the pressure into an electrical signal. The cells have built in thermistor so corrections can be made to data due to temperature fluctuations and the cells have long term stability (Geokon 2015). Figure 8-19 shows the earth pressure cell.

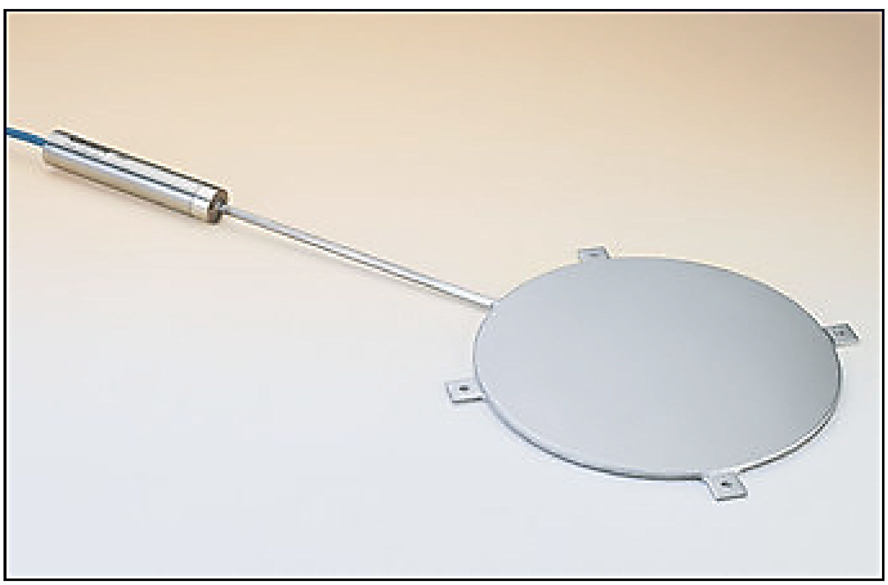


**Figure 8-19: Earth Pressure Cell (Geokon 2015)**

### 8.12.2 Linearly Variable Differential Transformer

The Linear Variable Differential Transformer (LVDT) can be placed in test pipes to measure the vertical and horizontal deflections of the pipe. The LVDT is an electromechanically transducer that can convert the rectilinear motion of an object to an electrical signal. The LVDT can read to a few millionths of an inch up to several inches. The LVDT assembly is illustrated in Figure 8-20; the casing is a stainless steel tubing with a magnetic shell surrounding the nickel iron core. The displacement is measured by the amount of voltage that is moving within the secondary windings. As the core moves due to outer displacement, the voltage in the windings change and the voltage change is read to measure the displacement. LVDTs can have unlimited mechanical life since there is no contact between any the core and the coil structure (TE Connectivity Corporation 2015).

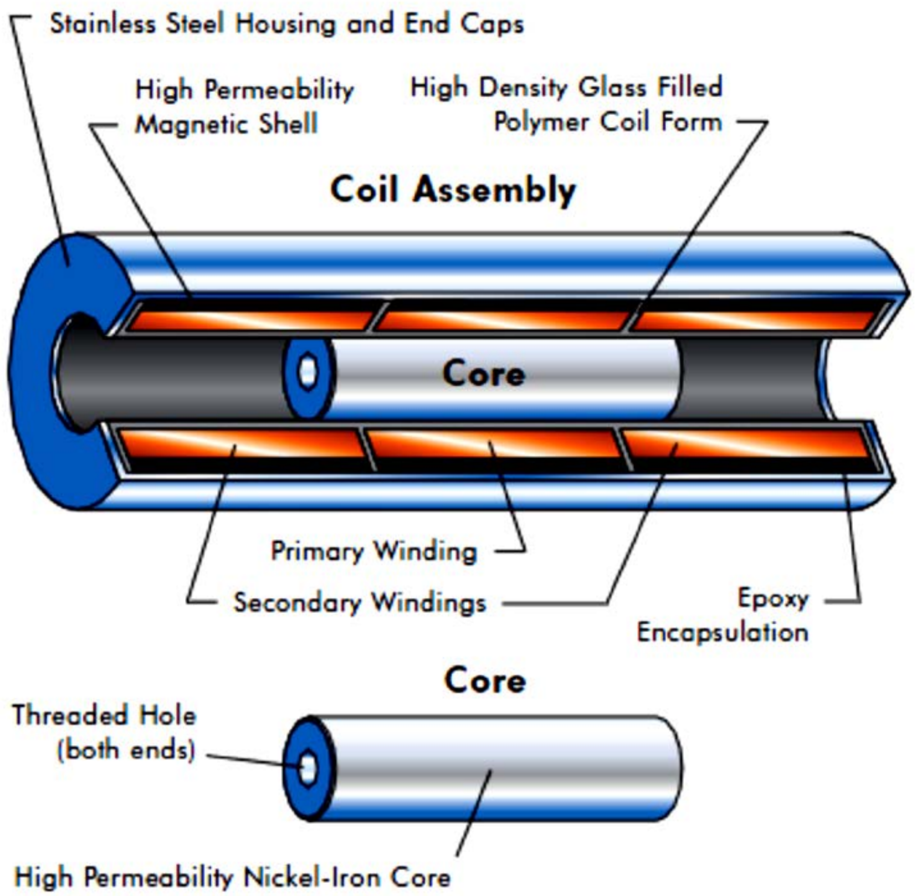



**Figure 8-20: LVDT (TE Connectivity Corporation 2015)**

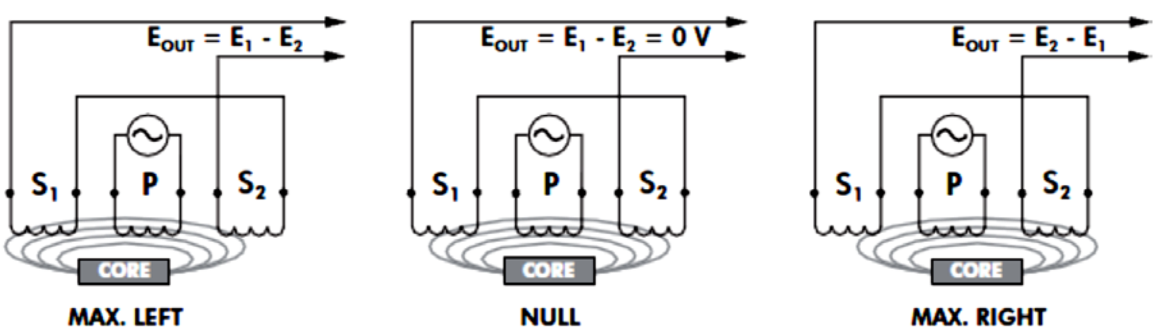



**Figure 8-21: Core Movement Effects on Readings (TE Connectivity Corporation 2015)**

8.12.3 Vibrating Wire Strain Gage

Vibrating-wire strain gages can be directly embedded into concrete to measure the strain in the concrete. The gauge is typically attached to rebar before it has cured in the concrete. The strain is measured by using a vibrating wire within the strain gage. The wire is tensioned between end blocks that can move as the concrete moves. The resonant frequency of the wire changes as the tension changes and the electromagnetic coil around the wire will read out the gage frequency. Figure 8-22 provides an illustration of the vibrating-wire strain gage.

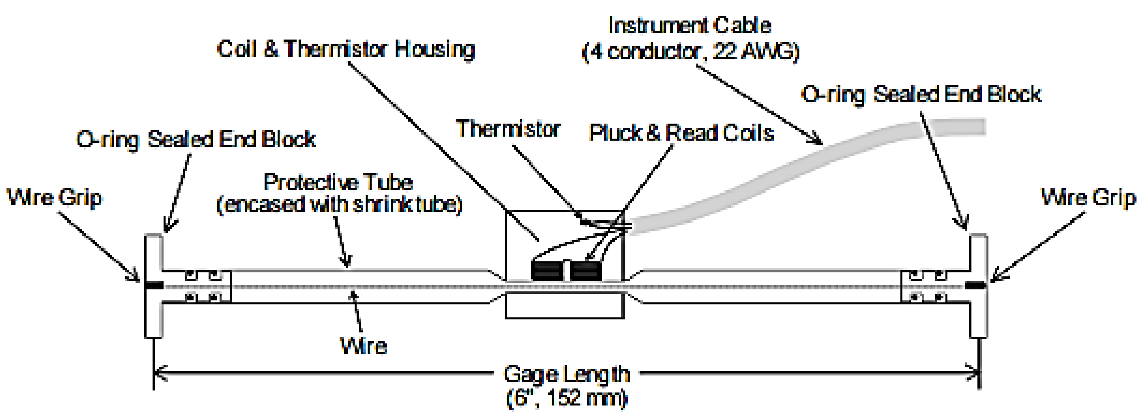



**Figure 8-22: Vibrating-Wire Strain Gage (Geokon 2015)**

## 8.13 Bedding and Backfill

### 8.13.1 Soil Classification

ASTM classifies soil according to ASTM D2487 - *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)*; AASHTO uses the AASHTO M145 – *Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes*. These are known as the Unified Soil Classification System (USCS) for ASTM and the AASHTO Soil Classification System (ASCS). The USCS soil breakdown is provided in Table 8-6. Gassman (2006) gives a table of the equivalent ASTM and AASHTO soil classifications.

# Table 8-6: Backfill Class and Quality (Gabriel 2008)

| Pipe Embedment Material | | | | | | | E, psi for Degree of Embedment Compaction | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| ASTM D 2321 | | ASTM D 2487 | | AASHTO M 43 | Min. Std. Proctor | Lift Placement | Dumped | Slightly <85% | Moderate 85%-95% | High >95% |
| Class | Description | Notation | Description | Notation | Density (%) | Depth | | | | |
| IA | Open-graded, clean manu-factured aggregates | N/A | Angular crushed stone or rock crushed gravel, crushed slag, large voilds with little or no fines | 5<br>56 | Dumped | 18" | 1000 | 3000 | 3000 | 3000 |
| IB | Dense-graded, clean manu-factured processed aggregates | N/A | Angular crushed stone orr other Class IA material and stone/sand mixtures; little or no fines | | | | | | | |
| II | Clean, coarse-grained soils | GW | Well-graded gravel gravel/sand mixtures; little or no lines | 57<br>6<br>67 | 85% | 12" | N/R | 1000 | 2000 | 3000 |
| | | GP | Poorly graded gravel, gravel/sand mixtures; little or no fines | | | | | | | |
| | | SW | Well-graded sands, gravelly sands; little or no fines | | | | | | | |
| | | SP | Poorly graded sands, gravelly sands; little or no fines | | | | | | | |
| III | Coarse-grained soils with fines | GM | Silty gravels, gravel/sand/silt mixtures | Gravel and sand with <10% fines | 90% | 9" | N/R | N/R | 1000 | 2000 |
| | | GC | Clayey gravels, gravel/sand/clay mixtures | | | | | | | |
| | | SM | Silty sands, sand/ silt mixtures | | | | | | | |
| | | SC | Clayey sands, sand/clay mixtures | | | | | | | |
| IVA** | Inorganic fine-grained soils | ML | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, silts with slight plasticity | | | | N/R | N/R | N/R | 1000 |
| | | CL | Inorganic clays of low to medium plasticity; gravelly, sandy or silty clays; lean clays | | | | | | | |

**Table 8-7: Equivalent ASTM and AASHTO Soil Classifications (Gassman 2006)**

| Basic Soil Type | ASTM D 2487 | AASHTO M 145 | ASTM D 2321 |
|---|---|---|---|
| Sn<br>(Gravelly sand) | SW, SP, GW, GP<br>Sands and gravels with 12% or less fines | A-1, A-3 | Class IB Manufactured processed aggregates, dense graded, clean<br>Class II. Coarse-grained soils clean |
| Si<br>(Sandy silt) | GM, SM, ML<br>Also GC and SC with less than 20% passing a No. 200 sieve | A-2-4, A-2-3, A-4 | Class III. Coarse-grained soils with fines<br>Class IVA: Fine-grained soils with no to low plasticity |
| Cl<br>(Silty clay) | CL, MH, GC, SC<br>Also GC and SC with more than 20% passing a No. 200 Sieve | A-2-6, A-2-7, A-5, A-6 | Class IVA: Fine-grained soils with low to meduim plasticity |

8.13.2 ASTM and AASHTO Requirements

Bedding provides firm support that is not too hard and levels out the pipe to the correct grade. Compacted granular materials should be spread and compacted evenly; Class 1, 11, or 111materials that can be used. However, AASHTO Section 30 stipulates a 1.25 inches (32 mm) the maximum particle size. When groundwater flow is expected then Class 1A materials should not be used; class III materials are adequate when moisture content is controlled. The class breakdown is provided in Table 8-6 (PPI 2008).

ASTM D2321 recommends a bedding layer of 4 inches and AASHTO recommends that bedding be compacted to a minimum density of 90% of the maximum dry density according to AASHTO T99. However, the bedding under the center third of the pipe diameter should not be compacted and should be a maximum of 6 inches (AASHTO 2012).

8.13.3 University of Massachusetts-Amherst Study

McGrath (1998) conducted a survey from multiple states on backfill materials. The study found that everyone that responded used granular materials as a backfill. However, the definition for granular material was different for each respondent. The names of granular filled used were select granular fill, granular backfill, gravel borrow and select materials. McGrath (1998) made a list of key points shown below.

- Three sponsors allow backfill with native material under some conditions.
- Compaction requirements generally vary from 90% to 100% AASHTO T-99.
- Eight of eleven respondents use controlled low strength materials (CLSM), also called flowable fill, under some conditions.
- Some sponsors specify minimum trench widths as low as the outside diameter plus 150 mm (6 inch). Most sponsors specify maximum trench widths (generally O.D. plus 0.9 m (3 feet)) or three times the outside diameter. Some distinguish between flexible and rigid pipe and some have trench dimensions dependent on the diameter.
- Ten of eleven require or recommend inspection during backfilling.
- Two of eleven require mandrel tests after backfill of flexible pipe.
- Eight of eleven require compaction testing.
- Two of eleven have specifications concerning groundwater control.

8.13.4 Chevron/PLEXCO Study

Petroff (1995) conducted a study on the installation techniques of flexible pipe. He stated that the embedment material quality, placement and densification, trench shoring techniques, groundwater, and workmanship effect the support of the pipe. Petroff (1995) claimed that the embedment material placement and densification was the most important factor to the performance of flexible pipe. A total of 28 pipes were tested and the Modulus of Soil Reaction (E') values were back-calculated. Values were obtained from 300 psi for dumped sand to 5000 psi for mechanically compacted crushed stone. For dumped crushed stone, large deflections were found and the E' did not exceed 500 psi. The density of the dumped crushed stone was supposed to be near maximum; however, the density ranged from 84 to 88 percent.

The most effective form of compaction was mechanical compaction with an impact tamper or plate vibrator. Light mechanical compaction increased the density of the crushed stone to excess of 90% that significantly affected the deflection. The difference between two tests was 1.5% average deflection for the mechanically compacted crushed stone to 6.4% for the dumped crushed stone. This scenario is when the in-situ is a wet, loose sand or soft clay.

The study investigated shoring and how shoring can affect the pipe support. For unshored trenches, it was concluded that unshored trenches are acceptable as long as mechanically compaction is used. However, shoring is typically used for the protection of life. Permanent shoring was concluded to be the most acceptable style of shoring since it did not affect the pipe support negatively and protected life. However, mechanically compaction was still significant in the performance of the pipe.

Temporary shoring can be done using trench shields or sheet piling. When the sheet piling is removed a void will remain between the trench and in-situ soil that reduces the side support. Using a vibratory extractor to remove the plate can cause a loss of ground and possibly liquefaction. The most applicable sheet piling is thin, non-corrugated sections. The most used shoring is trench shields. Typically, the trench shield will be placed and drag the shield along the trenchline for rigid pipe. However, this is not recommended for flexible pipe since it disturbed the in-situ soil down to trench grade and leaves a large void between the embedment and the in-situ soil. Petroff (1995) recommended that either the shield should be lifted in stages as the embedment is compacted since the embedment needs to be compacted against the in-situ soil or to place thin plates extending beneath the shield.

# SECTION 9

# TEST SETUP AND POTENTIAL SITE

## 9.1 Field Installation

Two plastic pipe test setups were investigated. The first is a field setup where the deflection is measured in an under-roadway field application. The second is a test frame that can test multiple pipe/soil scenarios. Pipes can be installed under new roadways to monitor deflection as done in the Phase 1 project. However, that approach is difficult since it must be put under a new road and typically deep soil cover is needed to actually put an adequate amount of stress on the pipe. One idea is to use the National Center for Asphalt Technology (NCAT) facility in Opelika, Alabama since the amount of traffic over the pipe could be controlled or measured. There are three possible locations at the NCAT facility to place the pipe: at the entrance to the test rack (Figure 9-2), at the exit to the test track (Figure 9-3, Figure 9-4), and buying a section of the test track under which the pipes would be installed. The locations are defined in Figure 9-1.

The NCAT field installations would give excellent live load analysis of the plastic pipe, especially since the loads would be overweight trucks consistently traveling over the pipe. However, this would require the NCAT facility to shut down for the time it would take to install the pipe that could take weeks. It does not seem feasible to make another entryway or exit so the NCAT facility could keep running as the pipes are being installed since there is very little room on the sides of the roadways and there are underground wires running next to them.  Also, the pipes would only be buried in shallow cover and not many different installations can be tested.


**Figure 9-1: NCAT Aerial View (Google Maps)**

**Figure 9-2: Test Track Entrance**

**Figure 9-3: Beginning of the Test Track Exit**

**Figure 9-4: Ending of the Test Track Exit**

## 9.2 Test Frame

A test frame could be constructed that would circumvent the field installation challenges. New pipes could be tested relatively quickly following established procedures to determine if they are adequate for cross-drain applications. The test frame could be placed somewhere on the NCAT facility but not under the test track. Most of the cost would go into the construction of the test frame but would be less for each installation than field test. Also, the apparatus could test multiple scenarios relatively quickly and have accurate control of key parameters that would affect the pipe performance. A possible location on the NCAT facility that is not being used can be seen in Figures 9-1 and 9-5.

The test frame is a better option the field investigation since it would be able to test pipes in a shorter amount of time. The field investigation would need multiple years to see if the deflection has increased. The test frame can simulate a more conservative scenario and if the pipe passes then it can at least handle a less conservative scenario. The test frame will need to be able to do multiple types of test.

**Figure 9-5: Possible NCAT Location for Test Apparatus**

# SECTION 10

# TEST FRAME DESIGN

## 10.1 Introduction

This section presents a preliminary test frame design. Detailed calculations are provided in Appendix F. The test frame is depicted in Figures 10-1 through 10-6. Note though that a majority of the completed apparatus would be below grade and partially supported by surrounding soil, which is not evident from the following CAD depictions.

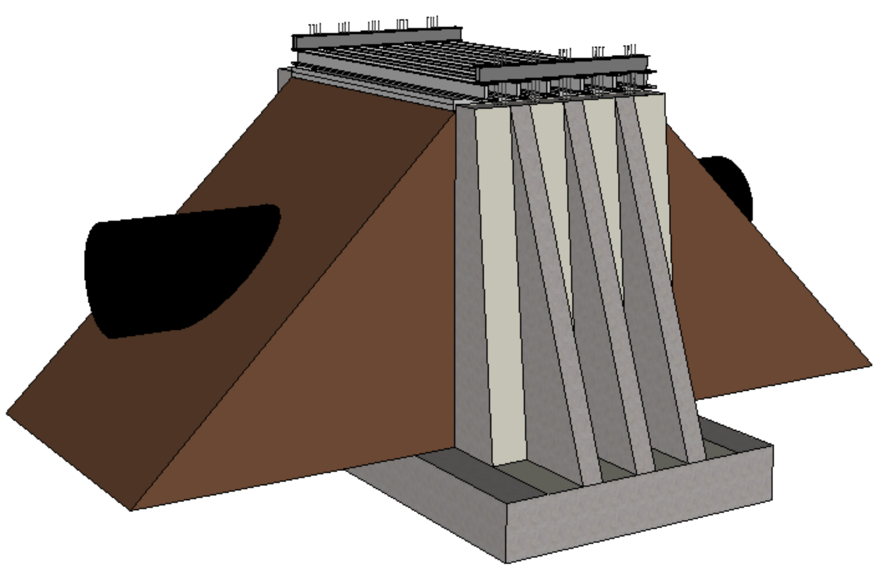


**Figure 10-1: Test Frame Rendering**

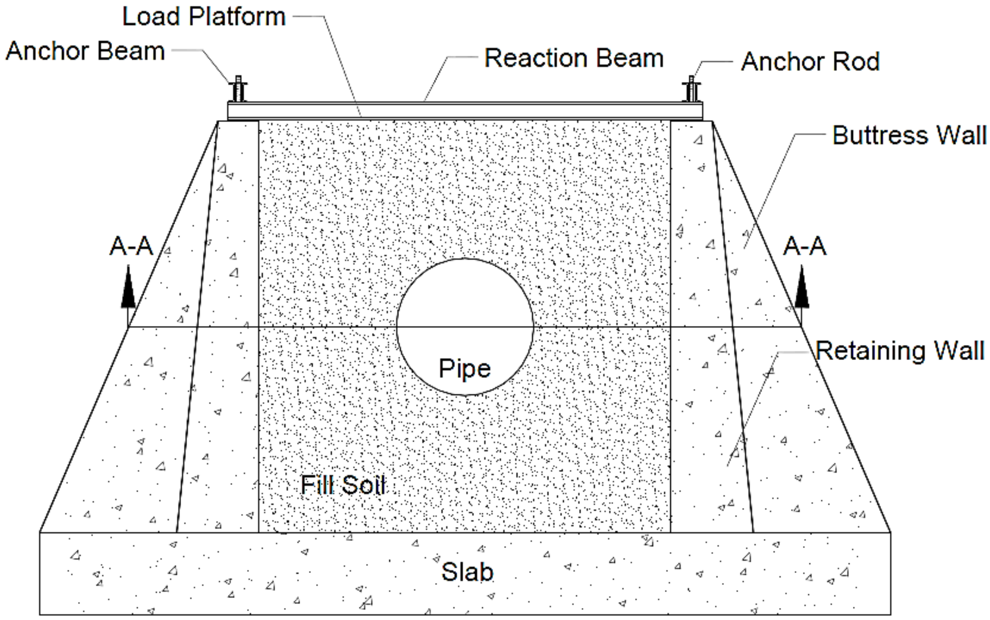



**Figure 10-2: Test Frame Transverse Profile**

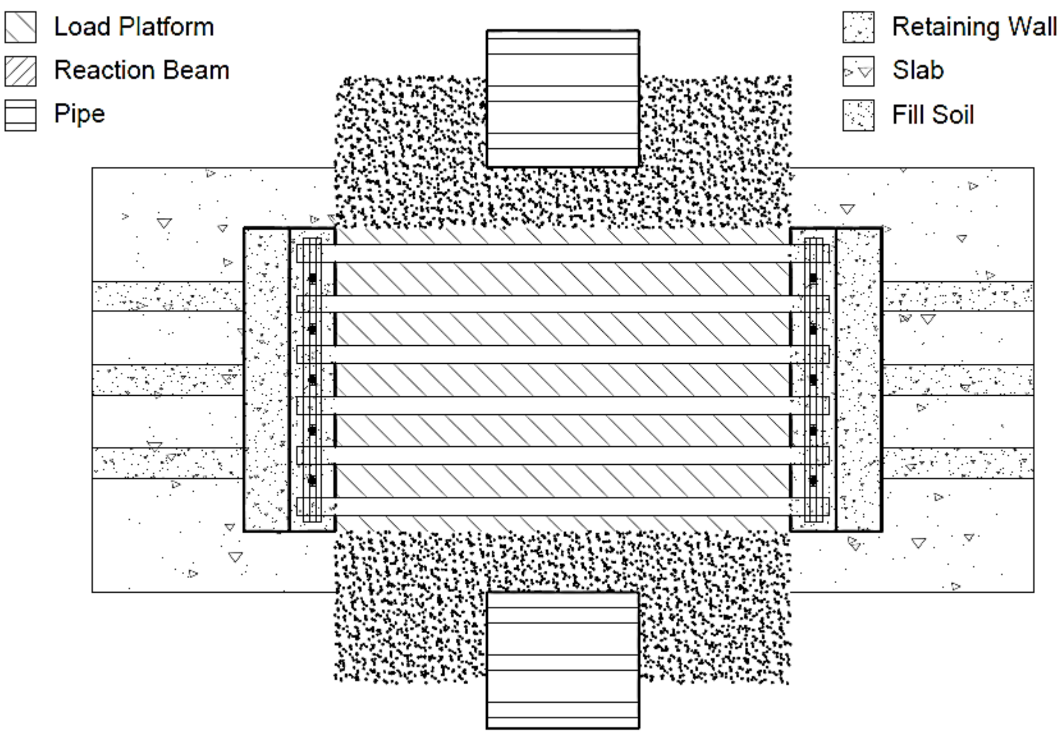



**Figure 10-3: Test Frame Plan View**

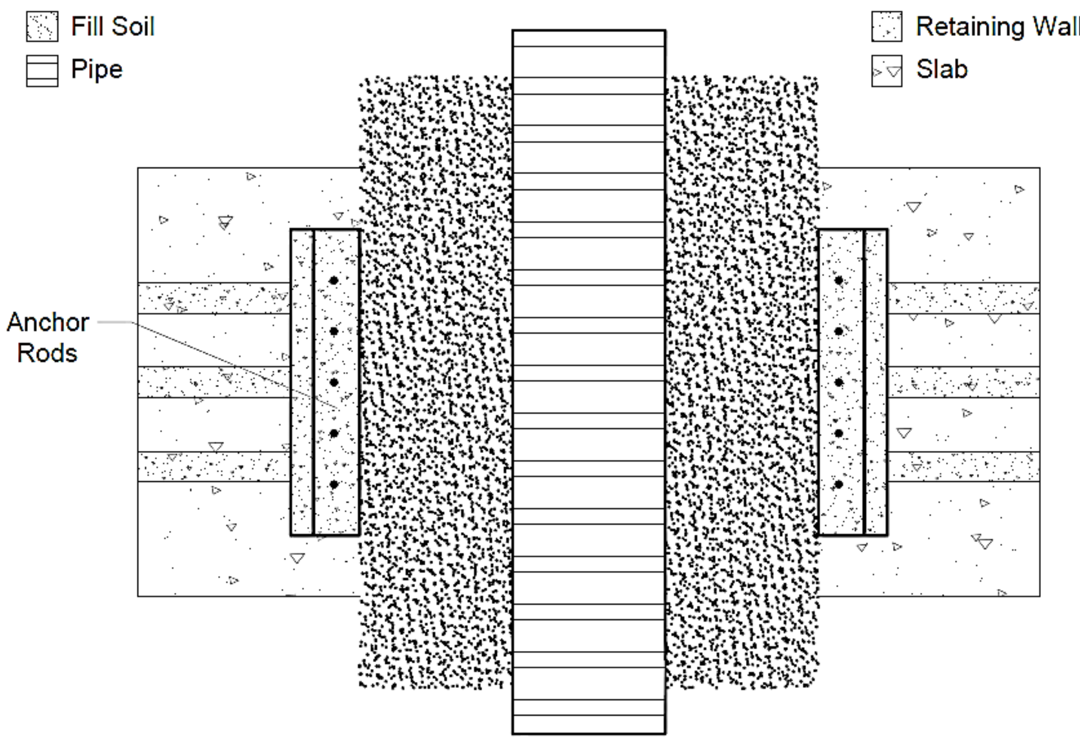



**Figure 10-4: Test Frame Section Plan View**

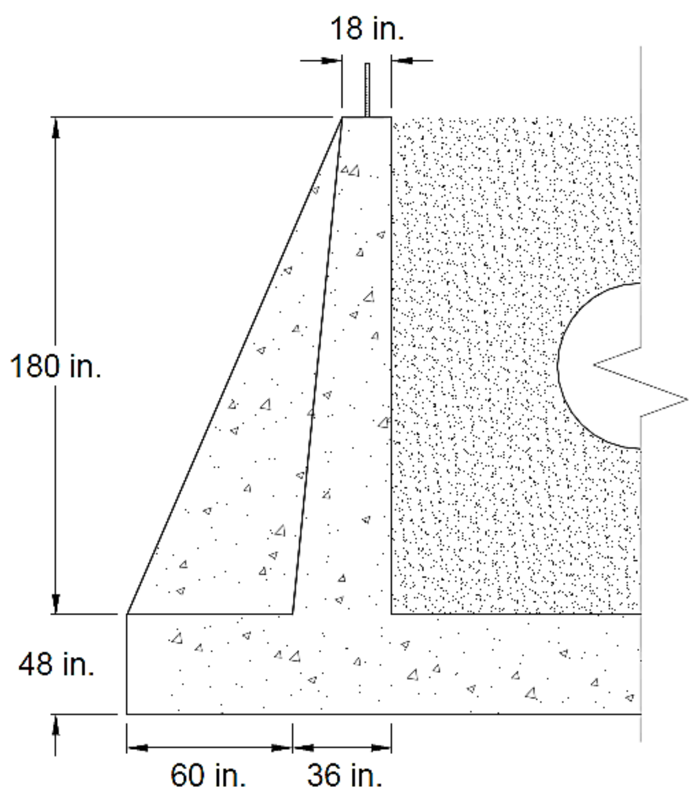



**Figure 10-5: Retaining Wall Section**

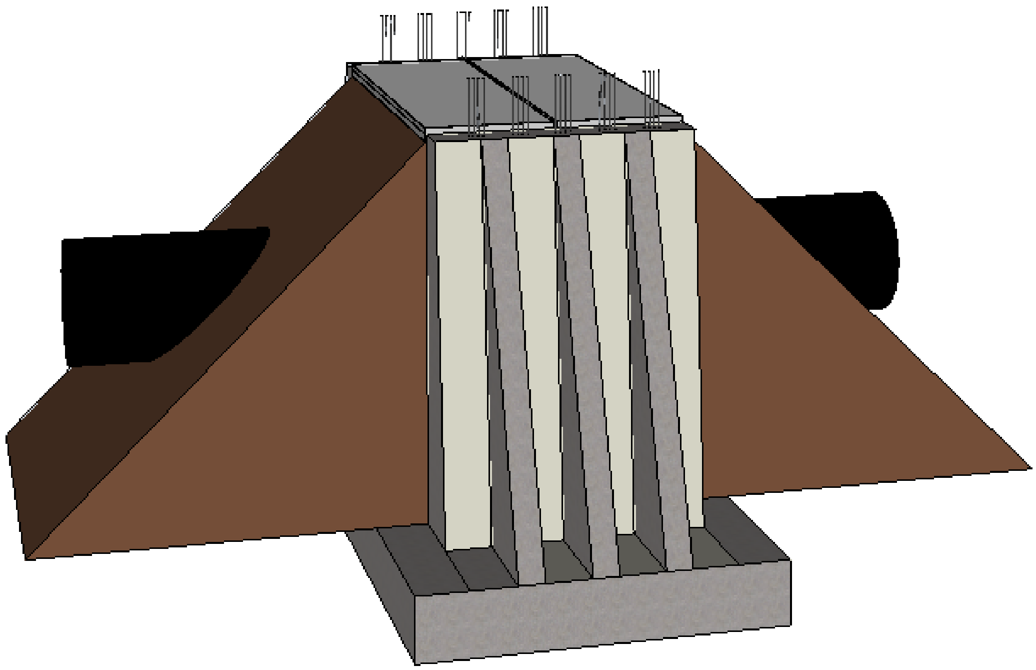


**Figure 10-6: 3D Test Frame Model**

### 10.2 Frame Dimensions and Rebar

The dimensions of the frame were chosen so that sufficient pipe length can be tested to provide results representative of real-world scenarios. Brachman et al. (2000) proved through finite element analysis that the farther the pipe is from the edges the less error will come from the boundary conditions, including friction forces from the walls. They stated that at least one pipe diameter of soil is necessary on each side of the pipe to negate the side wall friction. Since the span of the test frame is 15 feet between inner faces to inner faces of the walls, it is large enough to accommodate 60 inch diameter pipe sections. Also, with smaller pipes this will give even more soil space between the edge of the pipe and wall that will further diminish the error.

The AASHTO LRFD Bridge Construction Specification (AASHTO 2012) recommends the width of the trench to be at least 1.5 times the outside of the pipe diameter plus 12 inches and wide enough to safely operate compaction equipment on both sides of the pipe that the test frame size is adequate for a 60 inch pipe. ASTM D2321 recommends a minimum trench width of the greater of the pipe's outside diameter plus 16 inches or the pipe's diameter times 1.25 plus 12 inches, and therefore the test frame is adequate for a 60-inch diameter pipe.

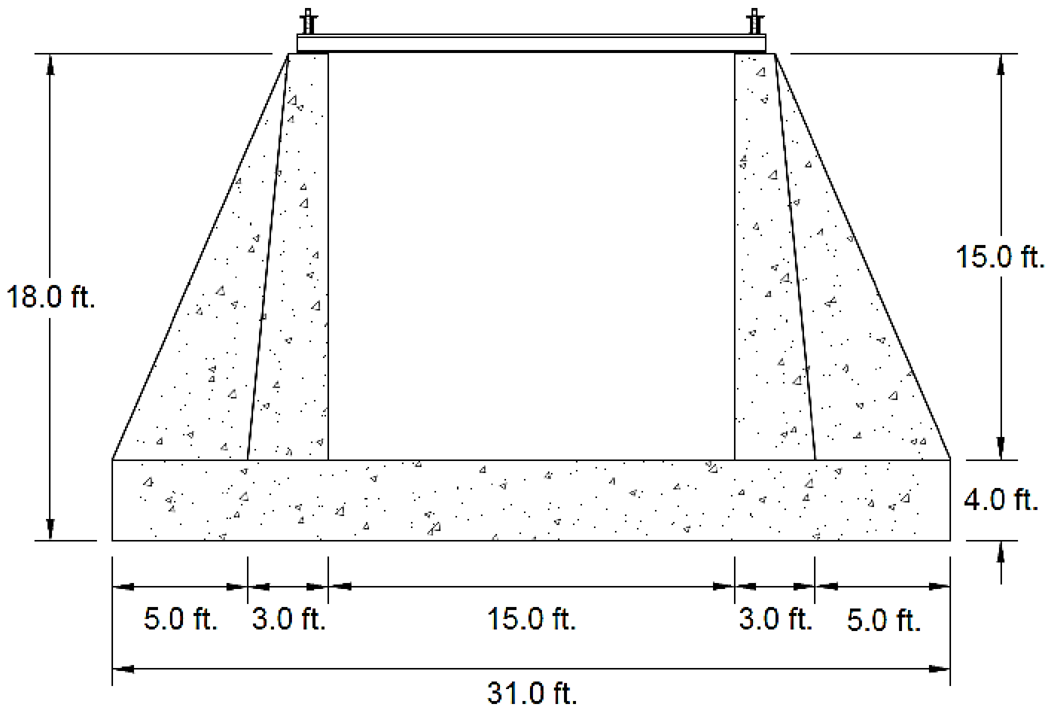



**Figure 10-7: Test Frame Profile Dimension**

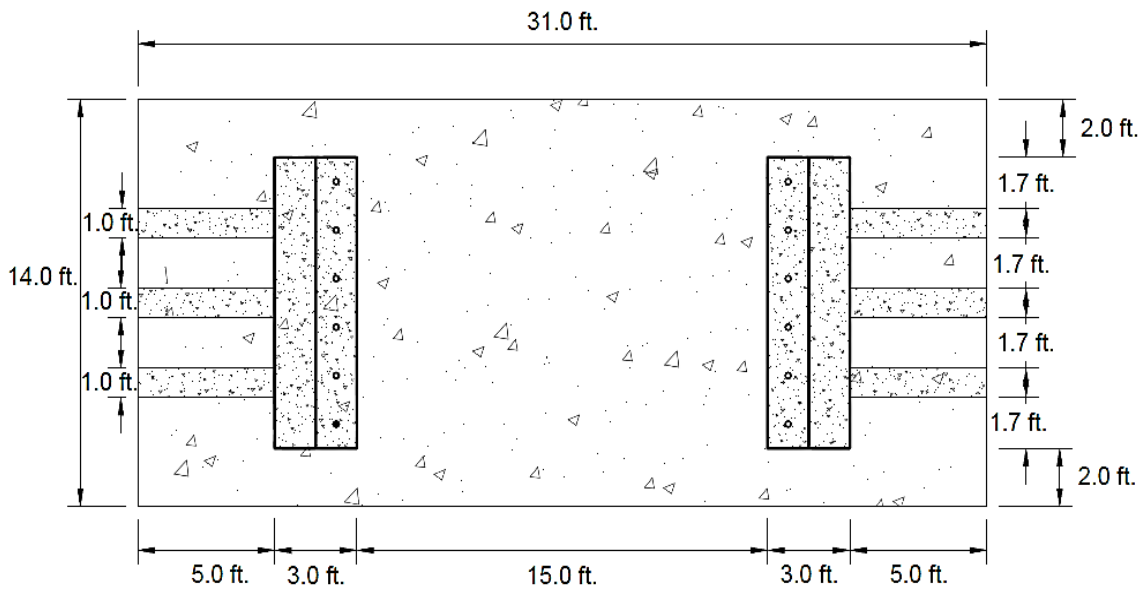



**Figure 10-8: Test Frame Plan Dimensions**

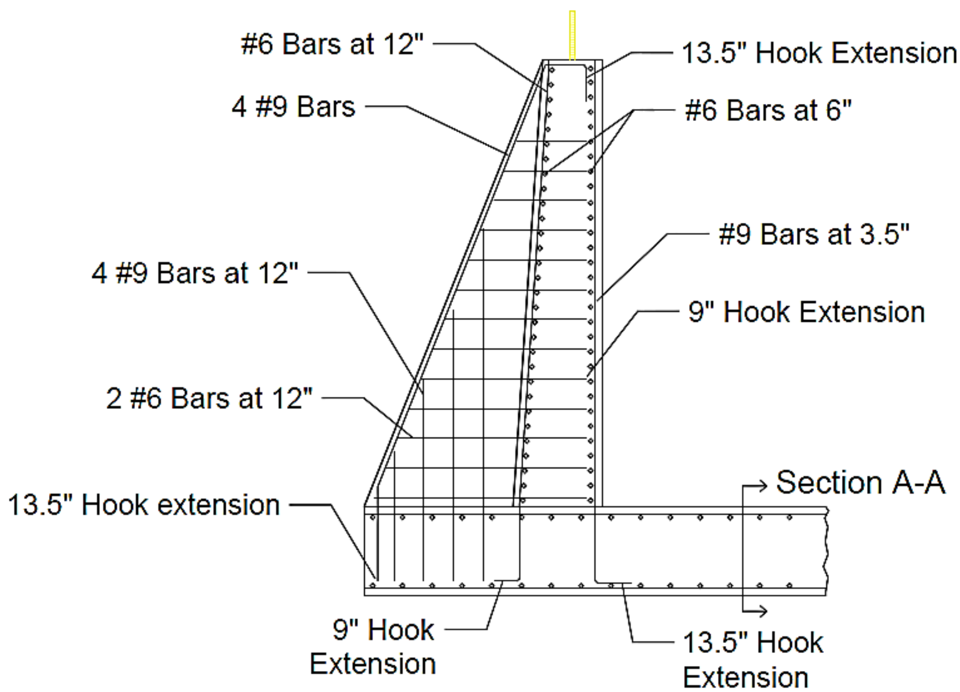



**Figure 10-9: Rebar Layout**

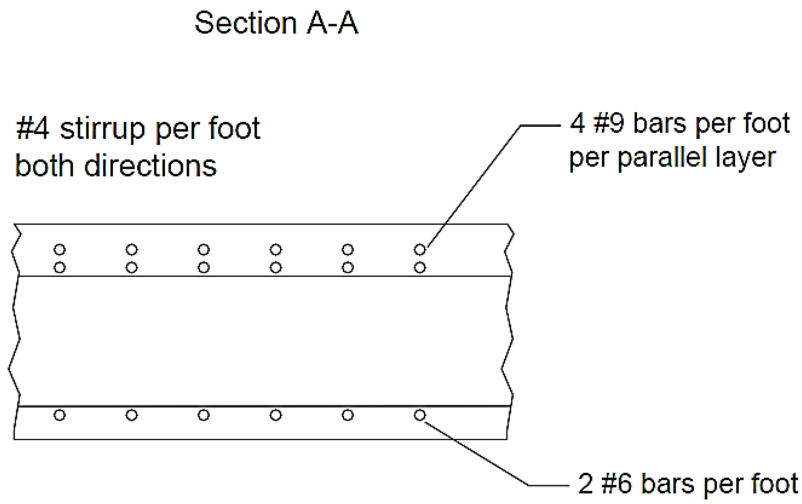



**Figure 10-10: Slab Rebar**

## 10.3 Boundary Conditions

### 10.3.1 Surface Friction

One of the boundary conditions that can cause error in the testing is the surface friction between the walls and soil. Since concrete is a typically rougher surface than steel it can cause more friction that will relieve some of the surface pressure farther down the testing apparatus. Brachman, Moore, and Rowe (2000) showed using finite element analysis how this surface friction can affect the results. They concluded that the effects of the surface friction are negligible if the angle of friction is less than five degrees. Abernathy (2012) concluded that bitumen coating would be adequate to reduce the angle of friction below five degrees.

The bitumen coating will need to be durable to handle the impingement of the backfill material especially for coarse gravel backfill applications. If the bitumen coating is not durable enough to handle the impingement then a protective layer will need to be placed. Brachman et al. (2001) designed a system to protect their friction agent as illustrated in Figure 10-11. Two 0.1 mm thick polyethylene sheets were used and were lubricated with silicone grease between them. The sheets were protected with thinner nonwoven geotextile and a 2 mm thick geomembrane.

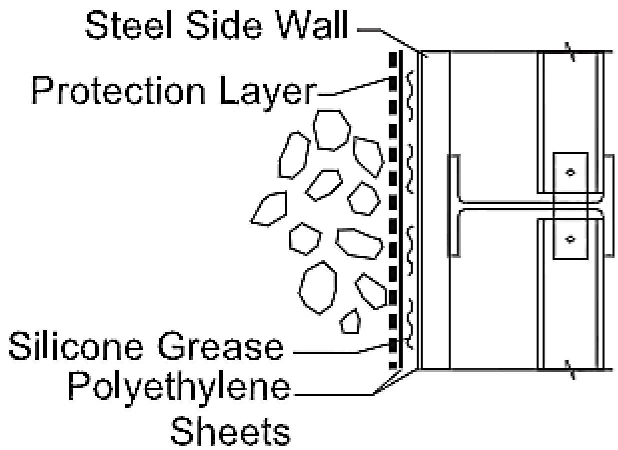



**Figure 10-11: Friction Agent Set-Up (Brachman et al. 2001)**

10.3.2 Wall Stiffness

Brachman et al. (2000) concluded that the best way to simulate boundary conditions is to use stiff walls on the edges. The stiff walls will simulate the actual conditions of buried pipes since typically there is so much soil on each side of the pipe that there is no deflection. As the vertical pressure increases the soil will try to move outwardly but the walls will keep it from moving, which will cause the lateral pressures similar to real life scenarios. However, Brachman et al. (2000) concluded that as the wall deflects outward the lateral stresses will decrease; even 1 mm of wall deflection can cause a 10% reduction of horizontal stress for soils with a 50 MPa soil modulus. They concluded that deflection needed to be less than 1 mm. From this conclusion, the testing apparatus will have walls similar to buttress retaining walls with the buttress limiting the deflection specifically at the middle of the wall but also at the edges. The buttress will make the walls extremely stiff making the deflection negligible.

## 10.4 Loading Platform

Abernathy (2012) concluded that a pressurized bladder would be the most efficient way to simulate the vertical pressures. The bladder will be pressurized and completely enclosed by the plate above, the soil below, and the concrete walls on the side. The bladder will try to expand and push down on the ground. Since the bladder is flexible, it will create a uniform load even though the soil surface will not be smooth. Brachman et al. (2000) and (2001) concluded that Buna N rubber is the best material to use for the bladder since it proved reliable at high pressures and can be seamed easily. Also, the material can be replaced easily and with minimum cost in case of failure.

The bladder can be pressurized using air or water. Air will be easier to compress and decompress so it is the recommended choice.

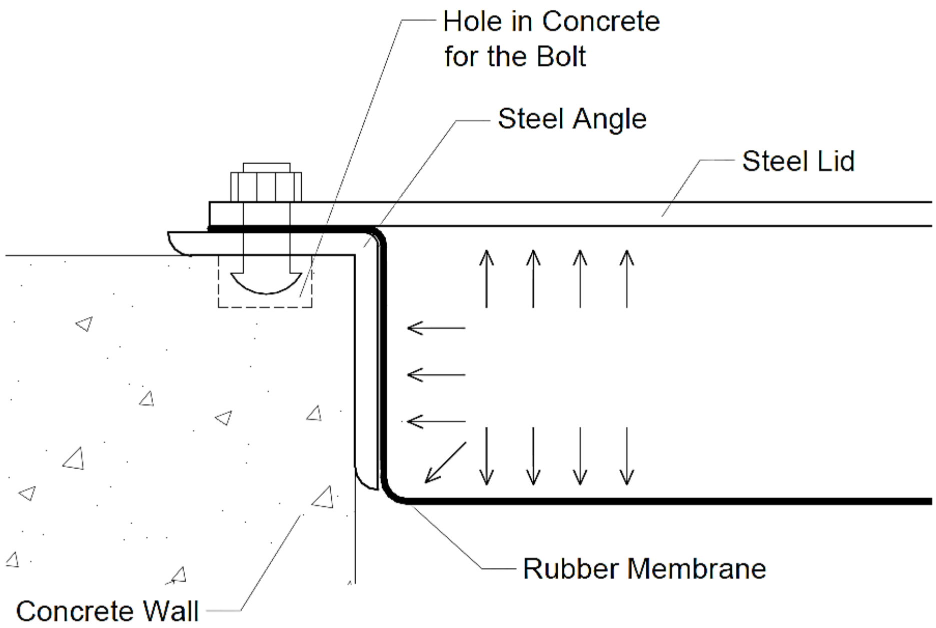



**Figure 10-12: Load Platform Components Detail**

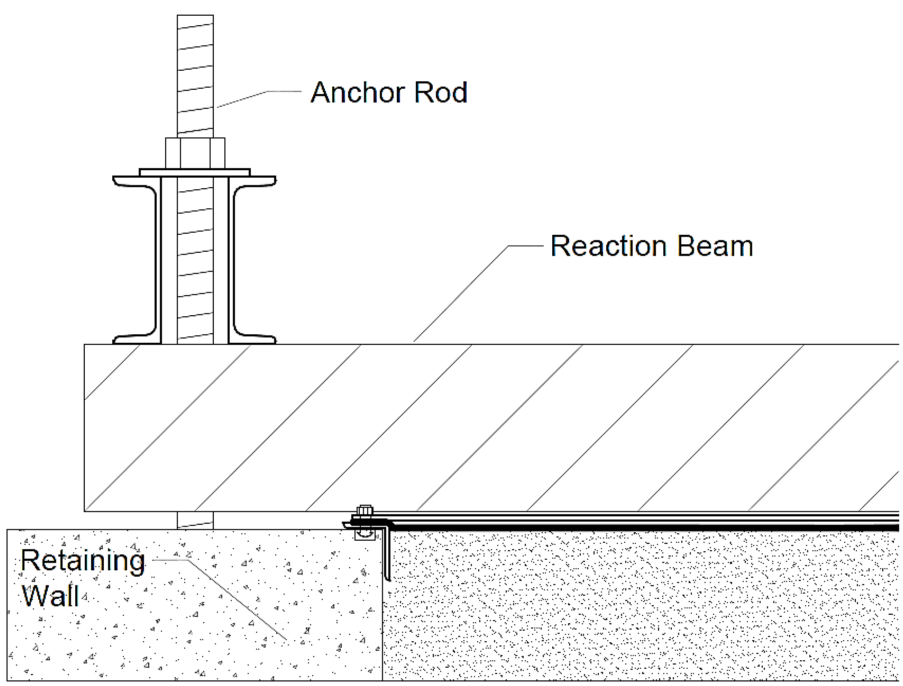



**Figure 10-13: Load Platform Components**

## 10.5 Frame Components

The bladder will be held down by four components: the steel plates, the reaction beams, the anchor beam and the post-tensioned anchors.

### 10.5.1 Steel Lid and Reaction Beams

The bladder will have a steel plate to restrain it from moving upwards.  The steel plate will be 1.5 inches thick A992 steel.  There will be six beams spaced 20 inches center-center with the edge beams spaced 20 inches center-edge of the plate.  The beams are W21x102 A992 steel.  The even spacing will minimize the stresses in the beams and plate.

### 10.5.2 Steel Angle

Steel angles will be used on the edge of the walls to assist the steel plate in containing the pressure from the bladder.  The steel plate and angle will be bolted together with the bladder between them to create a pressure seal.  The angle is better than the concrete since the corner of the edge will be curved to keep the bladder from being cut into and the angle will be consistently smooth across the whole edge, unlike the concrete, which will have bulges that can limit a perfect seal.  The angle will be a L3x2x1/4 and there will be a hole placed in the concrete to make room for the bolt holding the plate and angle together (Abernathy 2012).

### 10.5.3 Anchor Beams and Rods

The reaction beams must be anchored to the concrete. It is possible to anchor the reaction beams at the ends by connecting them with the anchorage tendons. However, the tendons would be anchored on the flanges, which becomes complex. Instead two beams at the ends of the reaction will provide the restraint. The anchor beams will be two channels welded together so that the anchor tendons can come through the middle of the channels that will put the load through the shear center.

The beams are double channels 2C15x33.9 made of A992 steel with a ¾-inch gap and will be welded every 20 inches. The plate holding the anchor bolt will be 1.5 inches thick. The length and width of the plate will be based on the supplier of the post-tensioned tendon. The anchor beam is illustrated in Figure 10-16. The rod will be embedded into the slab as seen in Figure 10-18. The two C-channels can be exchanged for thick HSS members if the HSS members meet the strength requirements.

The rod will be 270 ksi strength 6 fiber tendon with four tendons per anchor location. The locations of the tendons is shown in Figure 10-19. The anchor rods will be post-tensioned at the concrete surface. The anchor rods must be fastened to the anchor beams but still must be able to remove the anchor beam without the anchor rods. The current recommendation is to post tension the rods to the anchor beam; however, they would require the least amount of post tensioning force possible.

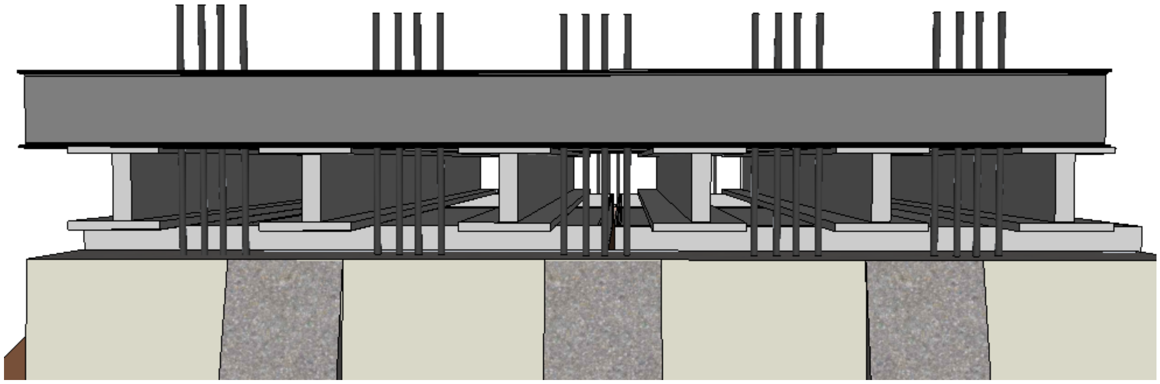


**Figure 10-14: Anchor Beam Rendering Side View**

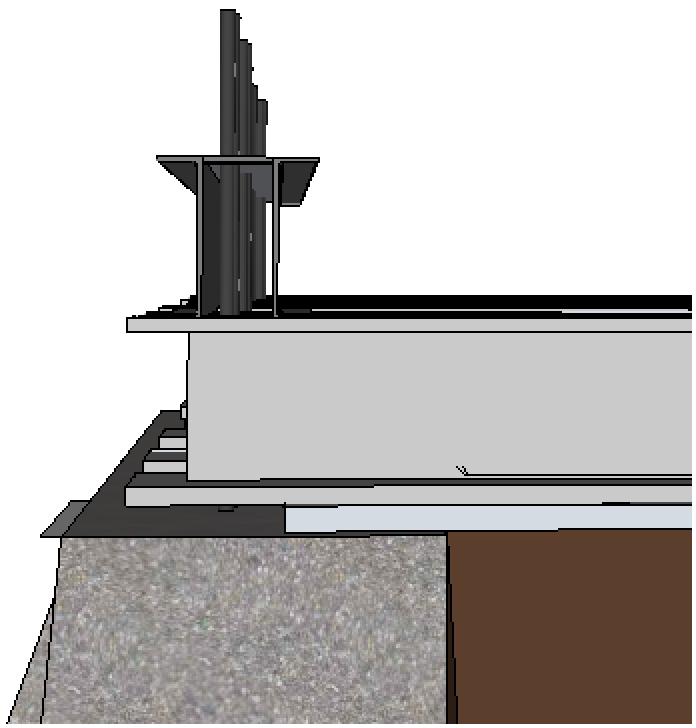


**Figure 10-15: Anchor Beams Longitudinal View Rendering**

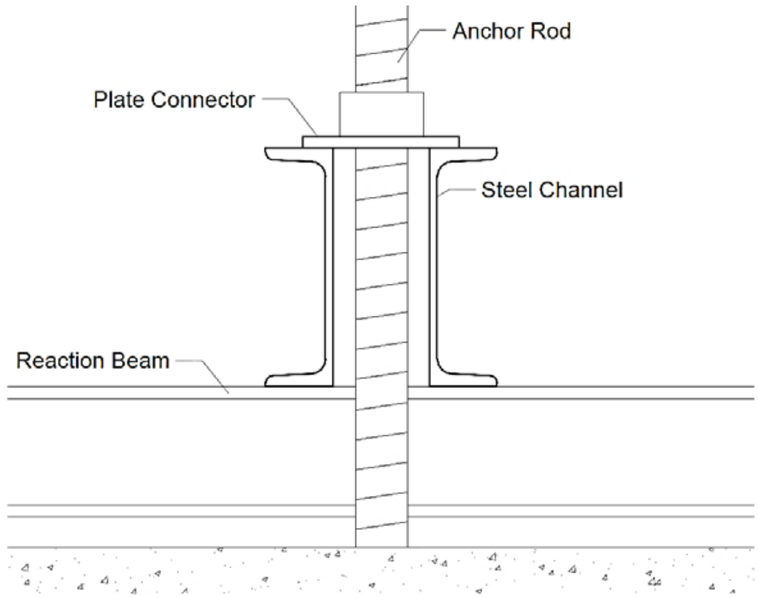



**Figure 10-16: Anchor Beam Longitudinal View**

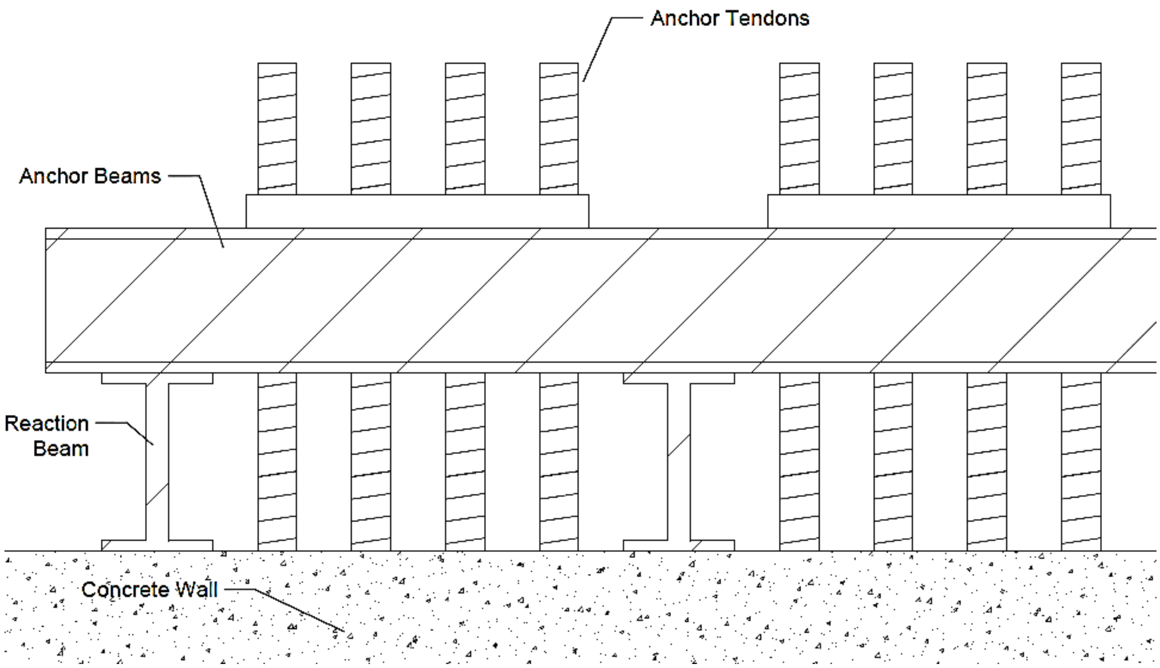



**Figure 10-17: Anchor Beam Side View**

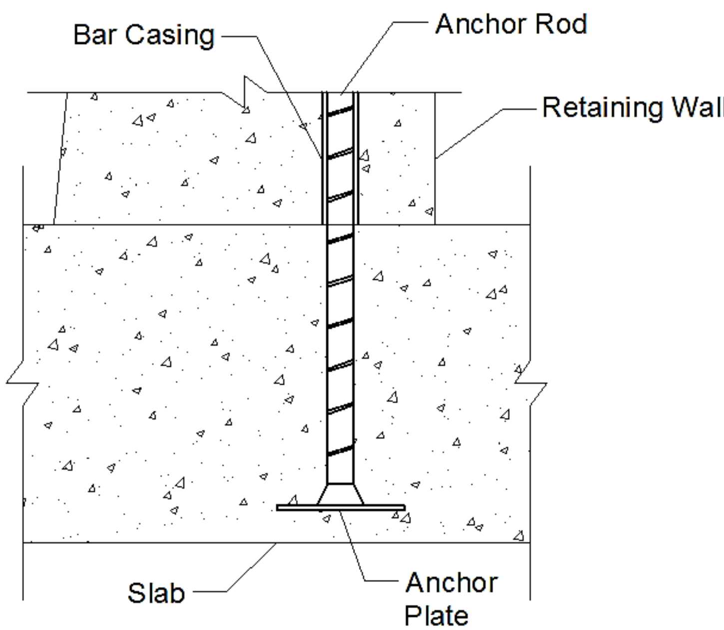



**Figure 10-18: Anchor Rod Embedded in the Slab (Abernathy 2012)**

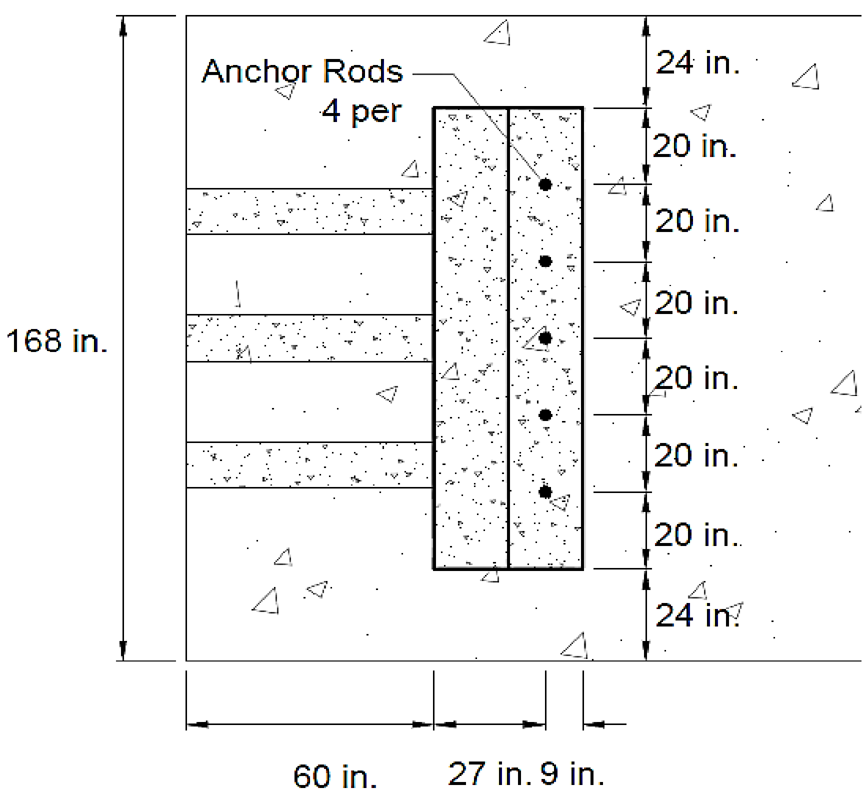



**Figure 10-19: Rod Location**

# SECTION 11

# TEST APPLICABILITY

## 11.1 Joint Testing

This section introduces tests that can be done with the testing apparatus, including joint testing, long term testing, testing at different temperatures, pulled-in-place testing, and pipe splitting. There are two types of joints for buried pipe: moment release (those designed to accommodate rotation that reduces the longitudinal bending moments close to zero) and moment transfer joints (those that limit rotation and transfer longitudinal bending moments from one pipe to the next) (Garcia 2012). An example of a moment transfer joint is a band connection and an example of a moment release is a gasketted bell and spigot joint. The gasketted bell and spigot joint will be focused on since it is most common for thermoplastic pipes. Moment release joints must accommodate the vertical shear force and rotation acting at the joint (Moore et al. 2012).

The testing apparatus will be able to accommodate joint testing. The most applicable approach will be to differ the bedding foundations around the pipe joint, which can be accomplished by not having enough or too much soil under the joint. The incorrect bedding will add stress to the joint. Two pipe segments of five feet each would be used with the joint at midway of the apparatus. The loads will be placed on the pipe with the pressurized bladder. However, the incorrect bedding will be a more realistic test than past tests where the pipe joint would not be tested with pipe-soil interaction in play. Typical incorrect beddings can be seen in Figure 11-1. Since the ends of the pipe are free it is possible to pressurize the pipe or have water run through the pipe to see if there any leaks in the joints.

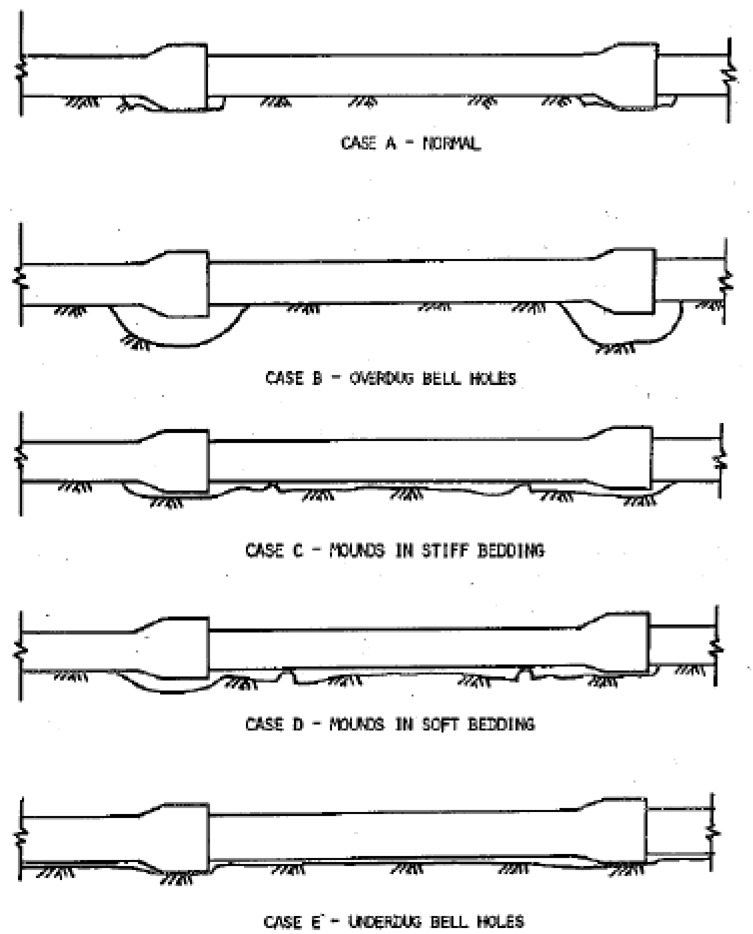



**Figure 11-1: Incorrect Joint Installations (Jeyapalan and Abdel-Magid 1987)**

### 11.2 Long Term Testing

Long term testing will be one of the most important applications of the testing apparatus since it is one of the main concerns when looking at plastic pipe. The testing apparatus can handle long term testing and should work smoothly since once the bladder is pressurized there is no more work needed other than to check that it has not lost any pressure. There could be some challenge for deep burial testing since the bladder can rupture when pushed closer to its limit. The testing apparatus would be tested under 45 psi of pressure, which correlates to approximately 50 feet burial depth with 127 pcf backfill. The pressure would be incremented to that point by pressurizing the bladder under 20 psi to 30 psi increments while holding the pressure for 20 minutes. Western Ontario had no problem with their bladder (Krushelnitzky and Brachman 2013).

The test can be conducted over months to see how the plastic pipe reacts but how long exactly to keep the test running depends of the objectives of each individual experiment. In a long term study of buried plastic pipe Ohio University stated that the vertical, horizontal, and circumferential shortening stabilized within 50 days after the completion of construction. However, a deeper look into their data found that larger pipes (up to 60 inches) stabilized in 90 days (Sargand et al. 2001). The recommended time would be 60 days for smaller pipes and 90 days for larger pipes.

### 11.3 Temperature Testing

Plastic pipe is affected by temperatures, which is significant since most of the test done on plastic pipe is around 73.4 $^{o}$F (23 $^{o}$C). The pipe system can be heated in the testing apparatus by circulating hot air through the pipe. The pipe would be heated before the load is placed on the pipe to keep the pipe from getting additional stresses from trying to expand, or the pipe could be heated afterwards.

### 11.4 Pulled-in-Place Pipe Bursting Testing

Pipe bursting is used when there is a pipe in the ground that is deficient, such as structurally deteriorated or hydraulically undersized. Typically the pipe would need to be excavated but pipe bursting gives an alternative method. Usually the method is used for gas, water, and sewer pipelines. Pipe bursting is most common in the sanitary sewer market in the United States but it is becoming more popular in the potable water market. The new pipe goes through the deteriorated pipe following a bursting head that breaks open the pipe with the new pipe following. The old pipe will remain as fragments surrounding the new pipe. An illustration of the pipe bursting operation is provided in Figure 11-2 (Simicevic and Sterling 2001).

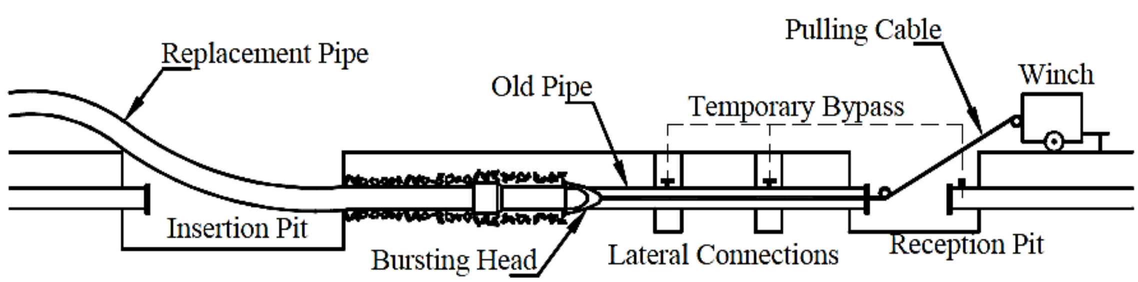



**Figure 11-2: Typical Pipe Bursting Layout (Simicevic and Sterling 2001)**

A typical pipe set up would be in position with the pipe and soil in the testing apparatus, and then the pipe bursting technique would be used to insert the new pipe. The load on the pipe would not be applied until the pipe bursting is completed. An example of the set up with the pipe bursting in action is provided in Figure 11-3. There can be a couple different situations where either a pipe that is the same size or a pipe that is bigger is replacing the deteriorated pipe called replacement test and upsize test, respectively (Lapos et al. 2007).

Pipe bursting is typically used to remove clay pipes, plain concrete pipes, and cast iron pipes since they are brittle. Steel and reinforced concrete pipe are not good for pipe bursting since they are flexible. PVC can be good if the right combination of pipe bursting and pipe splitting techniques are used. Typically HDPE is used as the replacement pipe since it can be fused together for long lengths, it is flexible and it can be used for many situations (Simicevic and Sterling 2001).

Lapos et al. (2007) used linear potentiometers (LP) with heavy plates at the end to measure the soil deflection. The pipe bursting procedure always has ground displacement since the bursting head is bigger than the original pipe. Future test could be performed on how the pipe reacts when the load is already applied while the pipe bursting action is occurring.

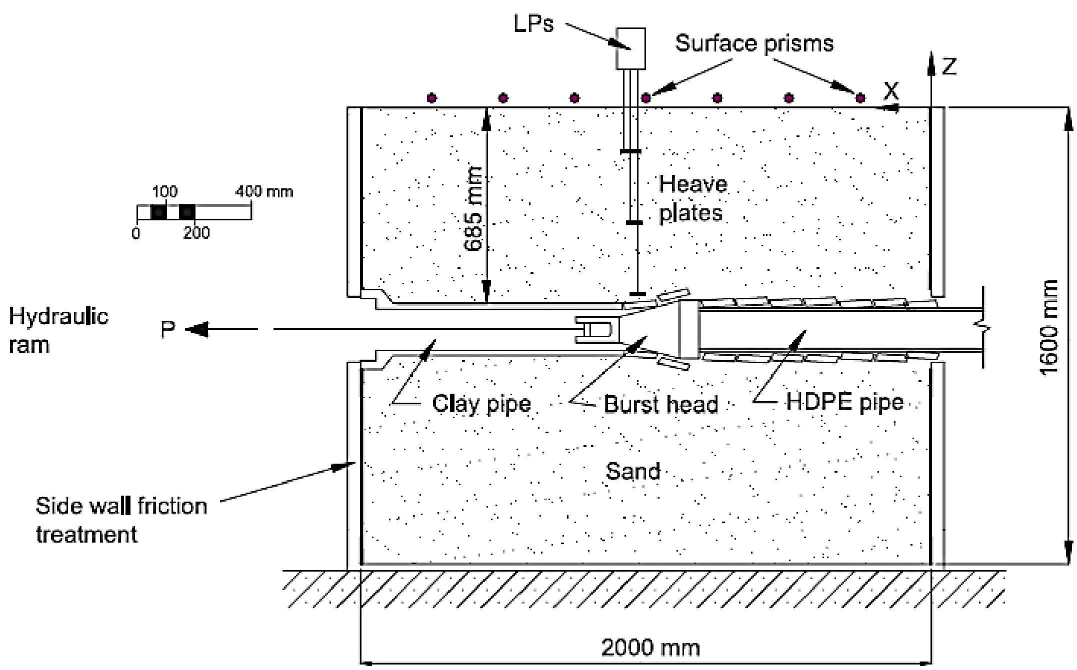



**Figure 11-3: Pipe Bursting Test Apparatus Configuration (Lapos et al. 2007)**

When the soil is homogenous with no close rigid boundaries the soil will deflect upward if it is at a smaller depth or if the pipe is in a trench, while at increased depths there is not much displacement. When there is weak soil below the pipe it will displace downwards. Figure 11-4 illustrates the displacement of the pipe in different soil conditions.

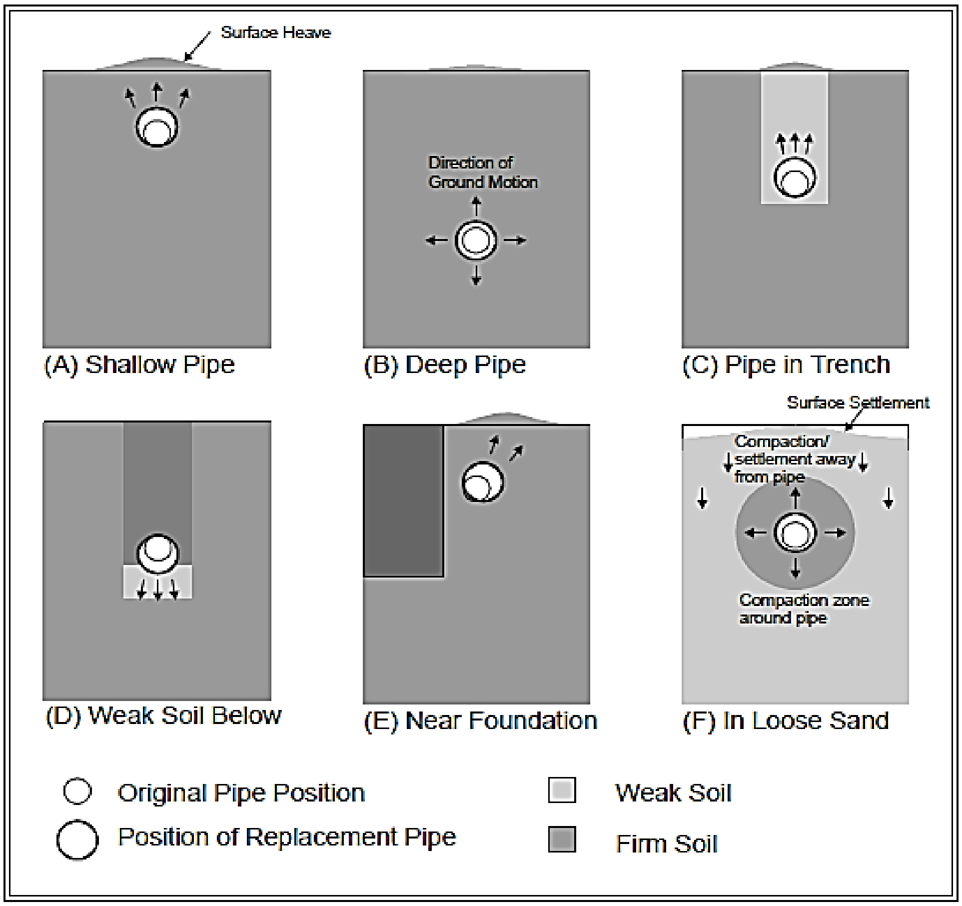



**Figure 11-4: Effect of Ground Conditions on Pipe Displacement (Lapos et al. 2007)**

## 11.5 Pipe Splitting

Pipe splitting is the replacement of an existing pipe with a new pipe by slitting the initial pipe down the longitudinal direction. Pipe splitting is needed for ductile pipes since they do not fracture using the pipe bursting technique. The same technique used in the pipe bursting section is used for the pipe splitting. The main difference is it uses a different head and the pipe would is not broken into pieces.  An example of the pipe splitting set-up is shown in Figure 11-5 (PPI n.d., Handbook of Polyethelyne Pipe).


**Figure 11-5: Pipe Splitting Head (PPI n.d., Handbook of Polyethelyne Pipe)**

# SECTION 12

# TEST PROTOCOLS

## 12.1 Pipe Stiffness

This section introduces recommended testing protocols for tests that could be performed using the testing apparatus, including pipe stiffness, long term testing, and testing at different temperatures. Recommendations on placement of the instruments are also presented in this section. While these tests provide a controlled testing methodology for plastic pipe, the manufacturers of the HDPE pipe have a quality assurance program that follows, at minimum, the NTPEP "Evaluation of HDPE (High Density Polyethylene) Thermoplastic Drainage Pipe Manufacturers".

Pipe stiffness is typically measured according to the parallel plate test. However, this does not give an accurate result for how much load a flexible pipe can resist when it is underground since the soil is part of the resistance. Typically all DOT's require the backfill soil to have at least 90% density (McGrath 1998). However, during the installation of the pipe, errors can occur that cause the backfill to not have as much density. Since there are problems that occur during installation, it is recommended to limit the pass/fail point to a more conservative value; there are two options given in this section.

The first option is to limit the percent deflection allowed to 2.5%. Typically most DOT's allow 5% deflection so this leaves a safety factor for incorrect installation. The recommendation is to use the soil and density that will most likely be used by that transportation department for the specific pipe. If there are multiple soils and densities that will be used, then either test multiple situations to get a good full understanding or test the most conservative situation. Once the pipe is installed the pipe will have an increasing load placed on it till it passes the 2.5% deflection limit and that load will be the maximum load that pipe is allowed within that specific backfill soil or within a higher class backfill.

The second option is to place the pipe in backfill soil that is recommended to be the most likely soil to be used by the department but with 5% less density than the density expected. The load will increase on the specimen until it passes 5% deflection and that will be the maximum load allowed for the pipe within that specific backfill soil or within a higher class backfill.

The soil would be placed in 6 to 12 inch lifts with each lift compacted. After the soil is placed the bladder will be put into position and pressurized in increments up with a max increment of 30 psi holding the pressure for 20 minutes. The middle section of the pipe will be measured since that is where the least amount of error occurs.

## 12.2 Long Term Testing

The final load on the specimen specified is up to each experiment with a maximum load of 60 psi allowed unless further testing shows the apparatus can handle the increased load. The recommended time of testing is 60 day since Ohio University concluded that in a long term study of buried plastic that the vertical, horizontal, and circumferential shortening stabilized within 50 days after the completion of construction. However, for larger pipe diameters (up to 60 inches) it is recommended that the maximum load be placed on the soil for 60 days (Sargand et al. 2001).

### 12.3 Temperature Testing

The test pipe can be heated by running hot air through the pipe from the two ends of the pipe until the desired temperature of the pipe is reached. The pipe will be heated before the overhead soil is placed to prevent more soil stresses on the pipe from the pipe heating. However, the pipe can be heated after the soil and load is placed to simulate real world scenarios where the pipe is heated by the liquid going within it after pipe placement. The soil would be placed in 6 to 12 inch lifts with each lift being compacted. After the soil is placed the pressurized bladder will be put into position and pressurized in increments up to a max increment of 30 psi and holding the pressure for 20 minutes. The middle section of the pipe will be measured since that is where the least amount of error will be.

### 12.4 Sheet Piling and Trench Shields Effect Testing

Petroff (1995) concluded that the trench shield can be incorrectly used when installing plastic pipe. When removed, the sheet piling and trench shields leave a void in between the in-situ soil and the backfill. The space allows the backfill to loosen lowering the density that supports the pipe. Both sheet piles and trench shields need to be tested when they are used correctly and incorrectly since a situation might occur where a pipe is installed incorrectly; the engineer would need to make a decision whether the technique is adequate or the pipe will need to be removed.

### 12.4.1 Sheet Piling

Sheet piling will need to be tested using both flat sheets and corrugated sheets. The test will compact the soil then raise it out after the pipe and backfill is fully installed by whatever method is normally used. There needs to be soil placed on both sides of the sheet pile so smaller pipes need to be used so one diameter of soil is on both sides of the sheet pile. The sheet pile will be placed then the soil on the outside of the pile will be placed and compacted. Then the soil and pipe on the inside will be placed and compacted. The sheet piling will be removed and then the load will be placed on the soil.

### 12.4.2 Trench Shield

There will be three different test for the trench shield installation. The first is to raise the trench wall in flights. The second is to raise the trench shield after the soil and pipe have been installed and compacted on both sides. The third is to pull the trench shield through with the soil and pipe installed and compacted on both sides.

The first test will install the trench shield and then place and compact the soil on the outside of the trench shield. Then the trench shield will be raised one foot and soil and pipe will installed and compacted on the inside of the trench shield at each increment. This will be done every foot all the way to the top. Then the load will be placed.

The second test trench shield will be installed and the soil on the outside will be installed and compacted. Then the soil and pipe on the inside of the trench shield will be installed and compacted. Then the trench shield will be raised out of the soil and the load will be placed.

The third test the trench shield will be installed and the soil on the outside will be installed and compacted. Then the soil and pipe on the inside of the trench shield will be installed and compacted. The trench shield will be pulled through the soil and the load will be placed.

**12.5 Soil Modulus Testing**

Soil Modulus testing will be done by applying the load until the deflection reaches 5%. The parameters will be analyzed and recorded, which will be focused on those listed shown below in the list and the equation but native soil type, native soil compaction density, and the water table will not be recorded since they do not apply to the testing. If the type of backfill and compaction normally used are known, then multiple tests can be conducted using the specific backfill and compaction to find an accurate Modulus of Soil Reaction for their specific design. Jeyapalan and Watkins (2004) gave a list of factors that can affect the soil modulus shown below.

- Native soil type
- Native soil compaction density
- Modulus of native soil
- Trench material type
- Trench material compaction density
- Modulus of trench material
- Size of pipe

- Pipe stiffness - soil stiffness ratio, $EI/(0.061E8r3)$
- Depth of cover
- Trench width – pipe diameter ratio
- Location of water table

Once the deflection is calculated the information is inserted into the Spangler Modified Iowa Equation as seen in Figure 12-1 (PPI 2008). The Modulus of soil Reaction is then back-calculated. The new data can be compared to the data in Appendix E.

(3-10)

$$\frac{\Delta X}{D_M} = \frac{1}{144}\left(\frac{K_{BED}\, L_{DL} P_E + \mathrm{K}_{\mathrm{BED}}\mathrm{P}_{\mathrm{L}}}{\frac{2E}{3}\left(\frac{1}{DR-1}\right)^3 + 0.061 F_S E'}\right)$$

and for use with ASTM F894 profile wall pipe as:

(3-11)

$$\frac{\Delta X}{D_I} = \frac{P}{144}\left(\frac{K_{BED} L_{\mathrm{DL}}}{\frac{1.24(RSC)}{D_M} + 0.061 F_S E'}\right)$$

**WHERE**

$\Delta X$ = **Horizontal deflection, in**

$K_{BED}$ = **Bedding factor, typically 0.1**

$L_{DL}$ = **Deflection lag factor**

$P_E$ = **Vertical soil pressure due to earth load, psf**

$P_L$ = **Vertical soil pressure due to live load, psf**

$E$ = **Apparent modulus of elasticity of pipe material, lb/in$^2$**

$E'$ =**Modulus of Soil reaction, psi**

$F_S$ = **Soil Support Factor**

$RSC$ = **Ring Stiffness Constant, lb/ft**

$DR$ = **Dimension Ratio, OD/t**

$D_M$ = **Mean diameter ($D_I$+2z or $D_O$-t), in**

$z$ = **Centroid of wall section, in**

$t$ = **Minimum wall thickness, in**

$D_I$ = **pipe inside diameter, in**

$D_O$ = **pipe outside diameter, in**

**Figure 12-1: Modified Iowa Formula (PPI 2008)**

### 12.6 Incorrect Installation

Incorrect installation would be tested by following the guidelines of the pipe stiffness testing section. However, the main difference would be to simulate incorrect installations. Also, the limitations of 2.5% deflection or lowering the density of the soil by 5% do not need to be followed unless extra conservatism is wanted. The pipe can be tested to 5% deflection all the other details of the installation need to be strictly restrained so they do not affect the results.

## 12.7 Instrumentation

### 12.7.1 Strain Gage

Vibrating wire strain gages will be used to monitor strains in the walls during the loading process. There are errors in the results as the walls deflect due to the relieving of the lateral stress. The deflections will be used to correct the test data from any errors.

Strain gages will be placed on the inner and outer parts of the test pipes to measure the local strain of the pipe at various locations. Since the geometry of the pipe will be different for the test the locations of the strain gages will change accordingly. However, overall the strain gages will be placed at the crown, shoulder, springline, haunch, and invert that can be seen in Figure 12-2 (Abernathy 2012).

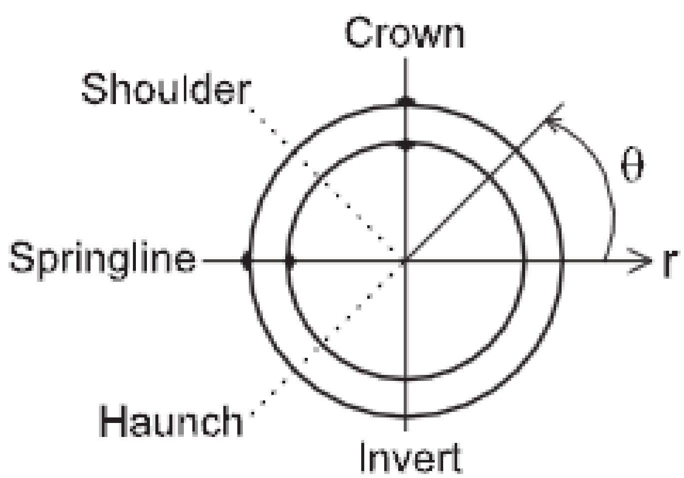



**Figure 12-2: Strain Gage Locations (Abernathy 2012)**

### 12.7.2 Earth Pressure Cells

Earth pressure cells will be used to define the force distribution through the soil in the testing apparatus. They will show how much the boundary conditions such as the side wall friction are effecting force distribution. The initial layout for the first test will be similar to Brachman et al. (2001) used that is showing in Figure 12-3 where the pressure cells will be placed on the edges of the testing apparatus to find the vertical and horizontal pressures acting at the edges. However, there will be four more pressure plates around the pipe to measure how much of the pressure is acting on the pipe. Also, there will be four pressure plates at the side edges to measure the vertical pressure change from the friction of the wall.

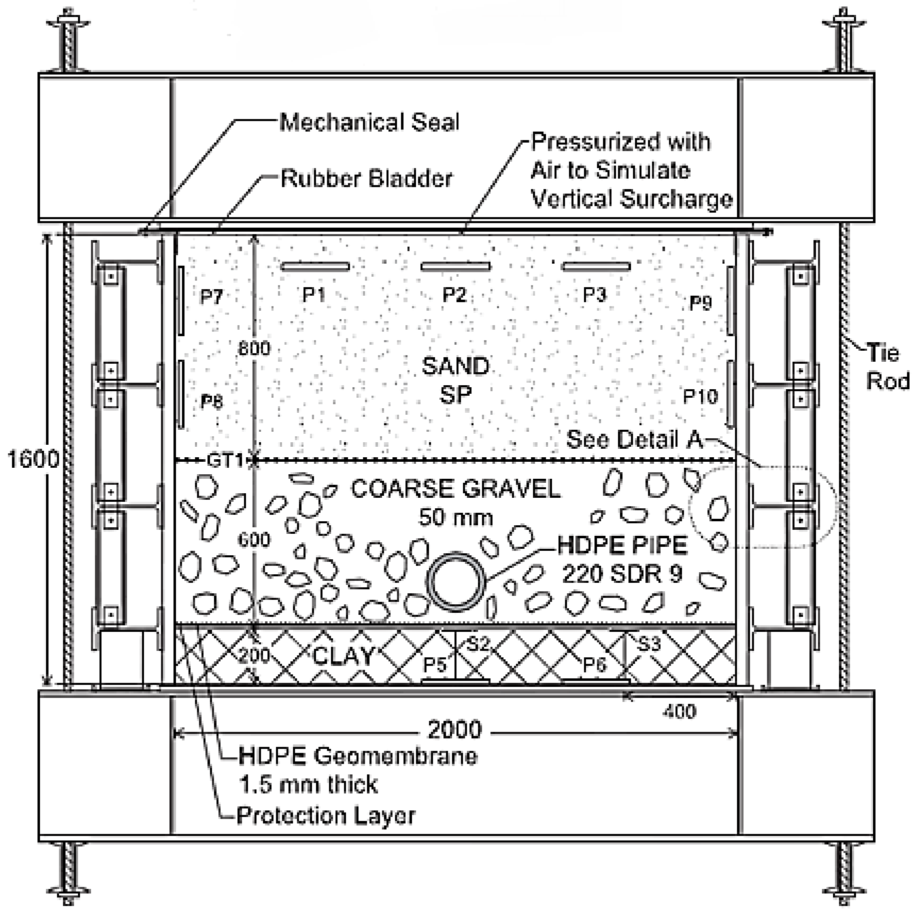



**Figure 12-3: Transverse Section of Western Ontario's Testing Apparatus (Brachman et al. 2001)**

12.7.3 Linear Variable Differential Transformer (LVDT)

Linear variable differential transformers will be used to measure the deflection of the pipe. The LVDT will be placed at the longitudinal center of the pipe to minimize location and length effects.

# SECTION 13

# SUMMARY AND CONCLUSIONS

## 13.1 Field Monitoring

### 13.1.1 Summary

The overall objective of the field monitoring portion of the research project was to assess the long-term performance of thermoplastic piping materials as cross-drains under highways. This objective was accomplished by completing three major phases: (1) comprehensive literature review, (2) deformation measurements and condition assessment of the Beehive Road thermoplastic pipes, and (3) inspection of regional thermoplastic installations.

The literature review consisted of an introduction to construction and installation practices necessary to achieve optimal thermoplastic pipe performance. This was followed with a review of thermoplastic culvert installation sites and studies that have been completed across the country reported by others, including research studies by the Pennsylvania Department of Transportation and Ohio University. These two studies provided insight into the performance of thermoplastic piping materials in differing environments.

The Beehive Road test site consists of five lines of pipe totaling over 1000 feet that were installed to assess the long-term performance of thermoplastic pipes as well as to evaluate the design and construction variables that affect their performance. These variables include pipe diameters, cover heights, bedding and backfill materials, and thermoplastic material. The inspections presented in this report are part of an ongoing accumulation of data to study the long-term performance of the site.

The third phase of the project consisted of inspections of other thermoplastic pipes that were installed throughout Alabama. Transportation engineers throughout the state of Alabama and the southeast were contacted in order to identify any known thermoplastic pipe installations. Engineers and construction officials involved in installation and design were interviewed to gain further details of the installation procedures.

### 13.1.2 Conclusions

The use of thermoplastic pipes as cross-drains under highways is still very limited. The installations studied have been in use for less than 15 years. The results of these investigations varied greatly, with some pipelines in acceptable condition while others were in poor condition. The Beehive Road pipes reveal only minor isolated deformation and joint integrity concerns thus far. The results from this task support the general conclusion from the Phase 1 project as well as studies by others that thermoplastic pipes have sufficient rigidity to be used for limited cross drain applications (such as restrictions on ADT and burial depth), but the development of the necessary rigidity is highly dependent upon precisely and consistently following the construction practices recommended by the manufacturers and governing industry organizations. Furthermore the ability of plastic pipes to maintain the required rigidity over the decades of typical design life is still in question. Because the pipelines studied under this task are still very early in their service life, inspections should continue to observe future changes. The varying results in the installations investigated for such a short service life, especially compared to the target service life, leaves open questions regarding the long-term success of thermoplastic pipes in cross-drain culvert pipe applications. Continued studies of these sites will provide more insight into the appropriate applications and appropriate limits for use of thermoplastic pipes.

#### 13.1.3 Recommendations

Beehive Road site monitoring should continue, the along with the Opelika, Montgomery, and Wetumpka sites. As these sites are monitored over a longer period of time, a more accurate representation of the long-term performance of thermoplastic pipes will evolve. Even though monitoring over the early parts of a pipelines life is important towards its long-term performance, a longer duration of monitoring will yield better analysis and conclusions. It is known that the installation of thermoplastic pipes is the key to successful use. It is recommended that DOT or construction officials closely follow recommended plastic pipe installation practices in order to achieve the best performance from the pipes.

### 13.2 Decision Algorithm

#### 13.2.1 Summary and Conclusions

Changes in federal legislation have led to increasing use of plastic pipes for transportation projects. Thermoplastic pipe offers considerable advantages over the conventional reinforced concrete pipe and the corrugated steel pipe. However, most state departments of transportation have limited experience with thermoplastic pipe and are hesitant to revise conventional selection policies.

In 2008, ALDOT contracted with Auburn University to investigate the field performance of plastic pipe in cross-drainage application. This marked Phase 1 of The Plastic Pipe for Highway Construction Project. Completed in 2011, the project addressed three distinct research components: a literature review, finite element modeling, and a field study.

The primary objective of the decision algorithm task was to develop a practical selection methodology that can be used by state and county highway engineers to consider all culvert pipe options equally and select the optimum material and class of pipe for cross-drainage application.

*The Specification for Culvert Material Selection* is comprised of checklists and a condensed summary of the material developed from the research. The checklists will allow for the quick selection or elimination of culvert material based upon predetermined site conditions. The accompanying condensed summary will serve as an on-the-go literature guide and represent fundamental information that has been extracted from the research. The checklists were created from durability concerns and installation requirements most commonly involved in the selection process. Crucial durability concerns include: sulfate content, resistivity, pH levels, abrasion, flammability, and ultraviolet radiation. Installation requirements include: minimum soil cover, maximum soil cover, culvert diameter, and presence of quality control personnel.

High density polyethylene, polyvinyl chloride, and polypropylene are viscoelastic materials. Viscoelastic materials exhibit a nonlinear stress-strain relationship that is time dependent. According to *Evaluation of Polypropylene Drainage Pipe*, “Polypropylenes exhibit higher tensile, flexural, and compressive strength and higher moduli than polyethylene” (Hoppe 2011). The mechanical properties of steel and concrete, on the other hand, are not significantly time dependent and are linear-elastic within the range of normal load applications. It has also been demonstrated that thermoplastic pipe may also be prone to slow crack growth through the pipe wall.

Thermoplastic pipes are generally considered to have good durability resistance; they are unaffected by the sulfate content, chloride content, resistivity in the soil, and hydrogen ion concentration of the surrounding soil and water. Reinforced concrete culverts, corrugated steel culverts, and corrugated aluminum culverts are all susceptible to these common forms of degradation factors. Plastic pipe also exhibits good abrasion resistance and will likely not experience the dual action of corrosion and abrasion. However, abrasion tests described in the literature have been based on using small aggregate sizes flowing at low velocities. Steel pipe is the most susceptible to abrasion, and aluminum pipe offers no improvement. The NCSPA recommends using non-metallic coatings over metallic coatings for increased abrasion resistance.

Plastic pipe is susceptible to high temperatures and ultraviolet radiation. According to the report *Evaluation of Polypropylene Drainage Pipe* published by the Virginia Center for Transportation Innovation and Research, “Polypropylene is highly susceptible to oxidation and undergoes oxidation more readily than polyethylene” (Hoppe 2011). All culvert materials are affected by fire and extremely high temperatures. The

National Fire Protection Association has given both polyethylene and polypropylene a rating of 1 (Slow Burning) on a scale of 0 to 4. The higher the rating, the more vulnerable the material to combustion. However, several departments of transportation have reported cases of the pipe ends catching on fire. The May 2017 fire that collapsed a portion of the I-85 bridge in Atlanta Georgia can serve as a recent extreme example of the combustibility of some thermoplastic pipes. AASHTO M294 2008 recommends protecting the exposed portions of polyethylene culvert pipes to protect against combustion and ultraviolet radiation.

Very few state departments of transportation recommend a fill height greater than 25 feet for high density polyethylene or polypropylene. However, thermoplastic industry organizations such as the Plastics Pipe Institute permit a fill height up to 50 feet for certain diameters of pipe. The cover height may be temporarily increased during construction to protect the culvert against heavy equipment. State transportation departments such New York and Florida have only recently begun to update selection policies and allow 60-inch diameter plastic pipe. Most state departments of transportation limit the diameter of a plastic pipe to 48 inches.

### 13.2.2 Recommendations

Thermoplastic pipe offers considerable advantages. However, most state departments of transportation are reluctant to install large-diameter thermoplastic pipes beneath major roadways with high volumes of heavy traffic since large diameter thermoplastic pipes developed for deep burial and high load applications are relatively new.  In an effort to counteract the hesitancy, the following recommendations are presented.

1. Perform a comparative economic analysis of the following culvert material for cross-drainage application: reinforced concrete, corrugated steel, corrugated aluminum, high density polyethylene, and polypropylene.
2. Continue monitoring the field performance of high density polyethylene culverts and polypropylene culverts installed in Alabama and the Southeastern United States.
3. Continue field testing and analytical research to determine the maximum allowable fill height for plastic pipes used as cross-drain culverts. Field testing should include standard highway construction equipment traffic.
4. Continue field testing to determine the maximum allowable pipe diameter of high density polyethylene culverts and polypropylene culverts.
5. Determine the effects of large bedload particles and high velocity flows on the inside of high density polyethylene culverts and polypropylene culverts. Current abrasion tests have used small aggregate sizes flowing at low velocities.
6. Research and test rehabilitation strategies specifically developed for plastic culvert pipes.

## 13.3 Test Protocol

### 13.3.1 Summary and Conclusions

A preliminary design of a testing apparatus that will simulate deep burial was developed.   The frame would provide the ability to test pipes in scenarios similar to real-world scenarios without having to conduct experimental installations under actual roadways. The frame was designed to be cost efficient and minimize the errors from the boundary conditions. The rubber bladder loading platform can provide accurate simulation of soil load. The construction cost was estimated at $100,000 and the installation cost for the pipe and soil is expected to be $10,500 per installation.

### 13.3.2 Recommendations

The test frame design presented herein is a preliminary design that would need additional professional evaluation and refinement. Finite element models could be developed to analyze the effect of boundary conditions on the results and to check the forces transferred into the test frame. The model could be used to define the length of the pipe required to avoid length effects. The test frame will need to be analyzed to determine the needed ground depth form a constructability and accessibility point of view.

In addition to rigidity and resistance tests, tests could be conducted to test the hydraulic performance of the pipe under loads. The pipe would be loaded with a joint in the middle. Then the pipe could be pressurized with water to see if any leakage happens. If there is water leakage, then analysis can be done to see how the water eroding the soil under the joint affects the performance of the pipe.

The use of the test frame as a possible benchmarking tool needs to be researched. If enough analysis is done with the test frame, finite element models could be created and validated to expand results from testing.

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# Appendix A: AET Services Laser Inspection Report

07/28/16

# Laser Inspection Report

## REPORT

## Auburn University

Completed for
Auburn University

**AET Services**

ROBOTIC INSPECTIONS

# AET Services

Robotic Inspections

# Project Summary

**Auburn University Project** **07/28/2016**

| ID | Upstream Node | Downstream Node | Pipe Size/ Type | Surveyed Footage |
|---|---|---|---|---|
| 222+00 HDPE | East | West | 36"/HDPE | 126.5 |
| 222+00 PVC | East | West | 36"/PVC | 128.4 |
| 223+00 HDPE | East | West | 36"/HDPE | 132.0 |
| 223+00 PVC | East | West | 36"/PVC | 127.5 |
| 224+00-54 | East | West | 54"/HDPE | 125.2 |
| 224+00-48 | East | West | 48"/HDPE | 123.2 |
| 230+00 | East | West | 36" PVC | 128.8 |
| 231+00 | East | West | 36" HDPE | 137.8 |
| | | | | |
| | | | | |
| | | Denotes Direction of | Inspection | |
| | | | | |
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| | | | | |

# Project Summary

**Inspection Date 07/28/16 Auburn University Cox Rd**

**All laser Inspection Software set for 5.0 % Overall Ovality deformation**

| | | |
|---|---|---|
| **222+00– HDPE** | **36”HDPE** | **West to East** |
| **126.5 Racking Defect ( Pipe Structure Instability)** | | |
| **222+00-PVC** | **36” PVC** | **West to East** |
| **138.1 (at Junction Box) 8.0% Ovality Deformation** | | |
| **223+00 HDPE** | **36” HDPE** | **West to East** |
| **No areas at 5.0% or greater** | | |
| **223+00 PVC** | **36” PVC** | **West to East** |
| **No areas at 5.0% or greater** | | |
| **224+00 –54** | **54” HDPE** | **West to East** |
| **122.0 6.8% Overall Ovality Deformation** | | |
| **224+00-48** | **48” HDPE** | **West to East** |
| **No areas at 5.0% or greater** | | |
| **230+00** | **36” PVC** | **West to East** |
| **No areas at 5.0% or greater** | | |
| **231+00** | **36” HDPE** | **West to East** |
| **No areas of 5.0% or greater** | | |

**AET Services utilizes Light Ring Laser Technology to produce laser imagery that is downloaded into Clearline Laser Software. We make every attempt to place camera in center of pipe and use optimum length for proper laser measurement. Utilizing this technique and the end treatments of these pipes make it difficult to darken the pipe for optimum imagery and the last 12-20 ft of Westerly Ends will not be laser inspected.**

**When reviewing the Ovality Report please visualize a line at the bottom points as there are ribs internally in HDPE pipe inherently making a wave type pattern.**

AET Services  Robotic Inspections
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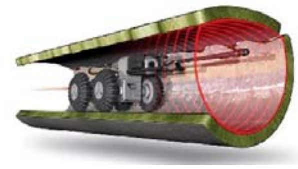
Liberty, SC 29657
1-800-990-8406

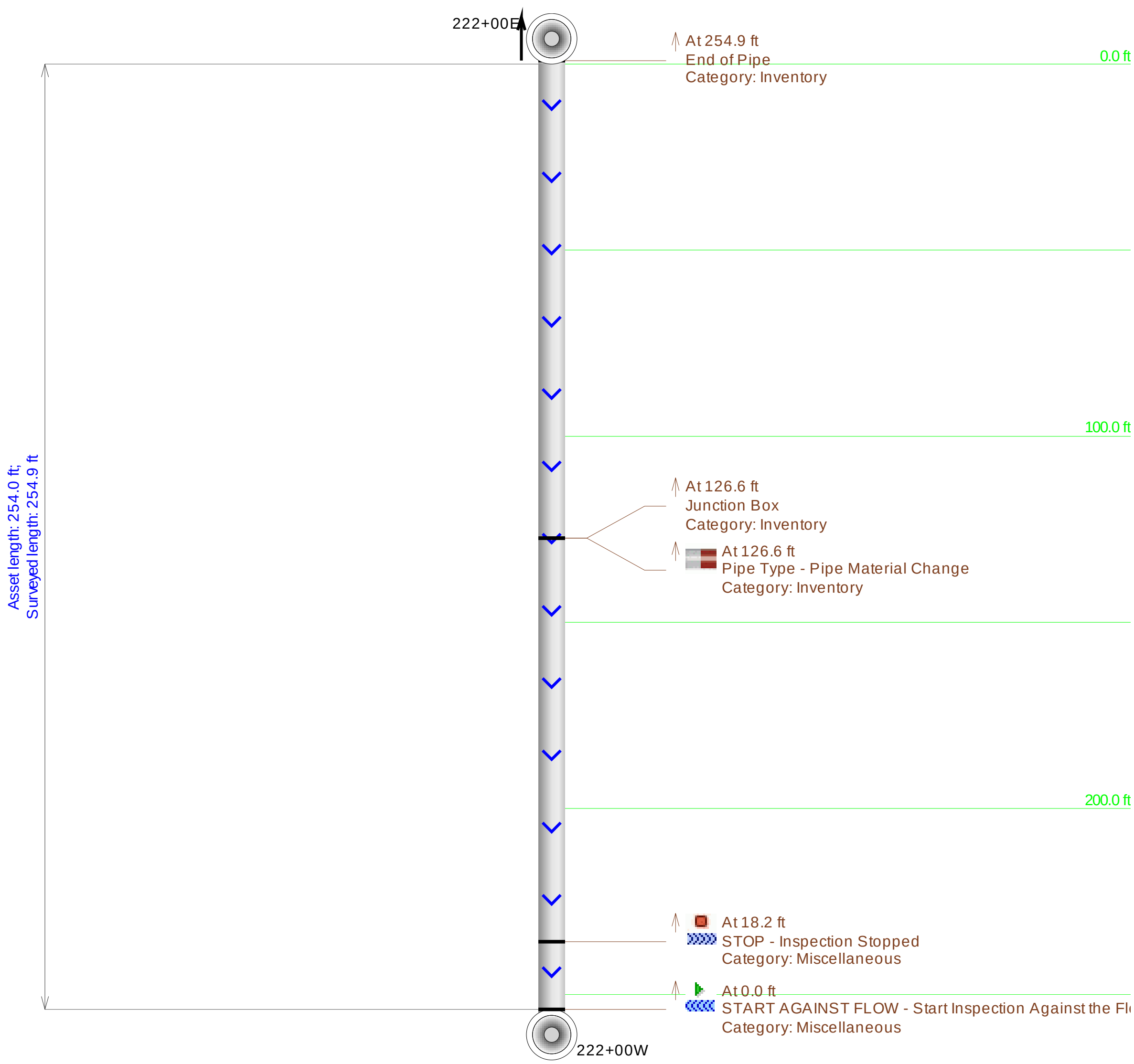


# TV Inspection with Pipe-Run Graph

| Project Name: | Mainline ID: | City: | Address: |
|---|---|---|---|
| Auburn University Research | 222+00W-E | Auburn AL | Cox RD |

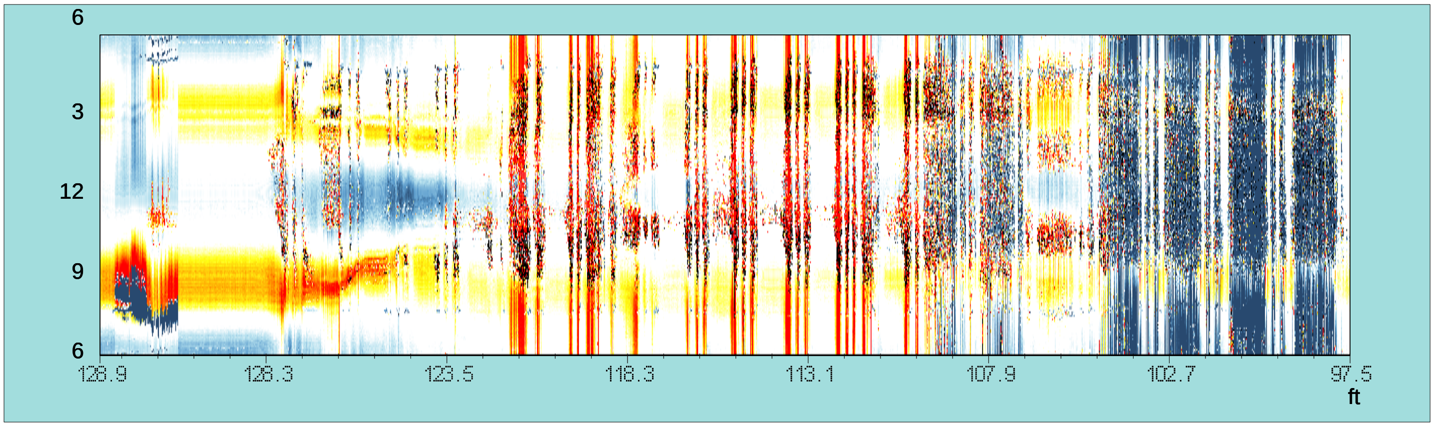
| Start date/time: | Pipe width: | Pipe height: | Pipe type: | Surface condition: |
|---|---|---|---|---|
| 7/28/2016 | 36 | 36 | Polyethylene | Asphalt |

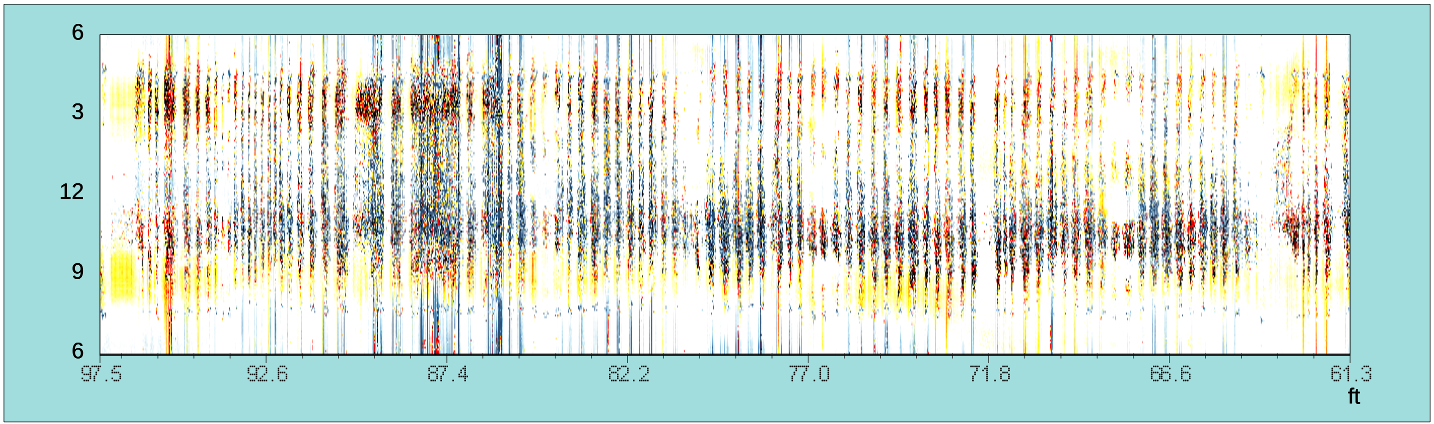
| Direction: | Surveyed footage: | Weather: | MediaLabel |
|---|---|---|---|
| Upstream | 254.9 | Dry | |

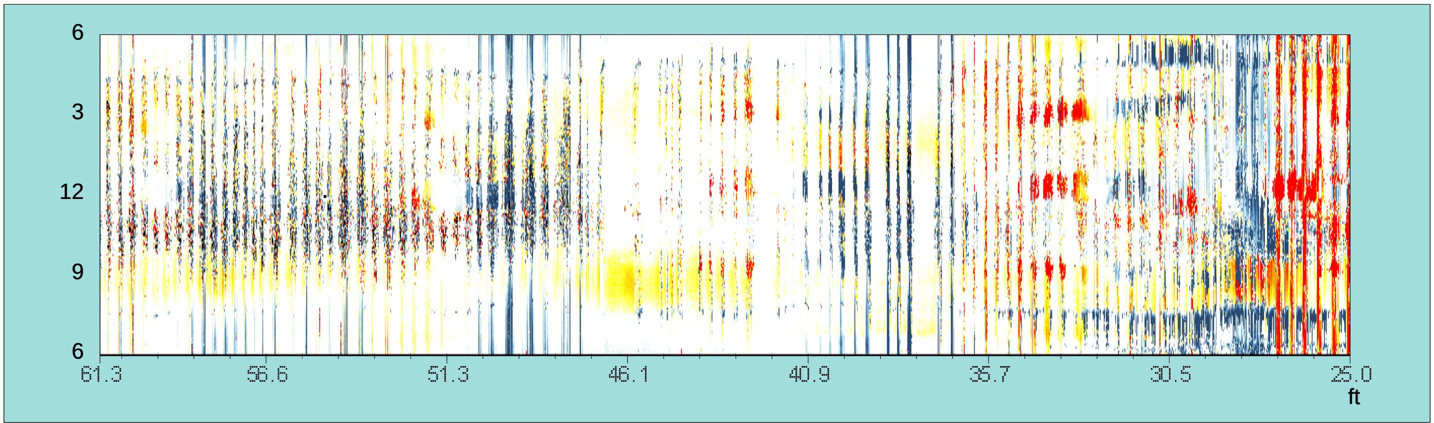


# Flat Analysis Report

| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 222+00HDPE | Date | 8/3/2016 |
| City | Auburn, AL | | | Material | HDPE |
| Start No | 222+00W | Finish No | 222+00E | Pipeline Length | 130ft |
| Location | Downstream | Location | Downstream | Internal Diameter | 35.87in |



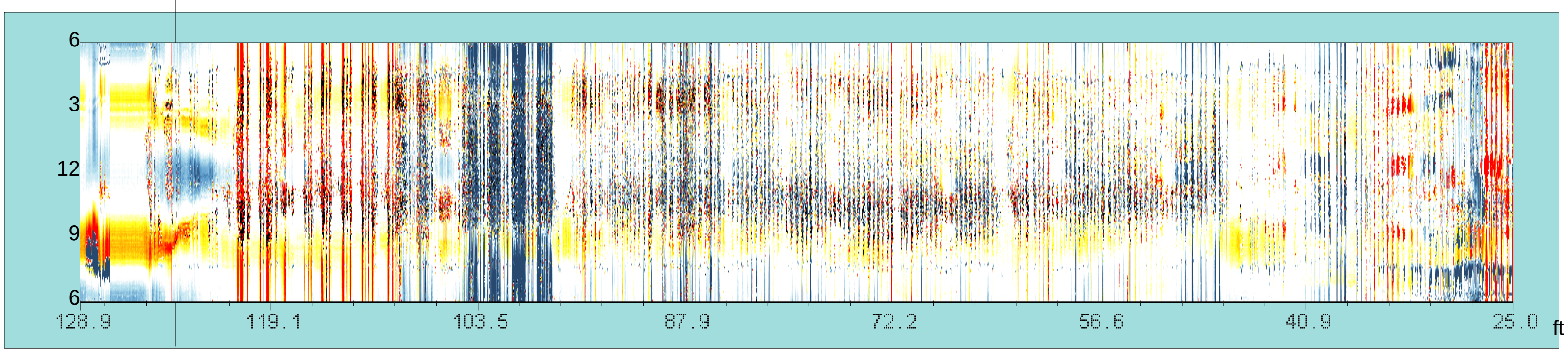

Comments

| r | | ø | r | | ø |
|---|---|---|---|---|---|
| 10.0% | 1.8in | 5.0% | -3.3% | -0.6in | -1.7% |
| 6.7% | 1.2in | 3.3% | -6.7% | -1.2in | -3.3% |
| 3.3% | 0.6in | 1.7% | -10.0% | -1.8in | -5.0% |
| 0% | 0.0in | 0% | | | |

# Flat Observations Report

| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 222+00HDPE | Date | 8/3/2016 |
| City | Auburn, AL | | | Material | HDPE |
| Start No | 222+00W | Finish No | 222+00E | Pipeline Length | 130 ft |
| Location | Downstream | Location | Downstream | Internal Diameter | 35.87 in |

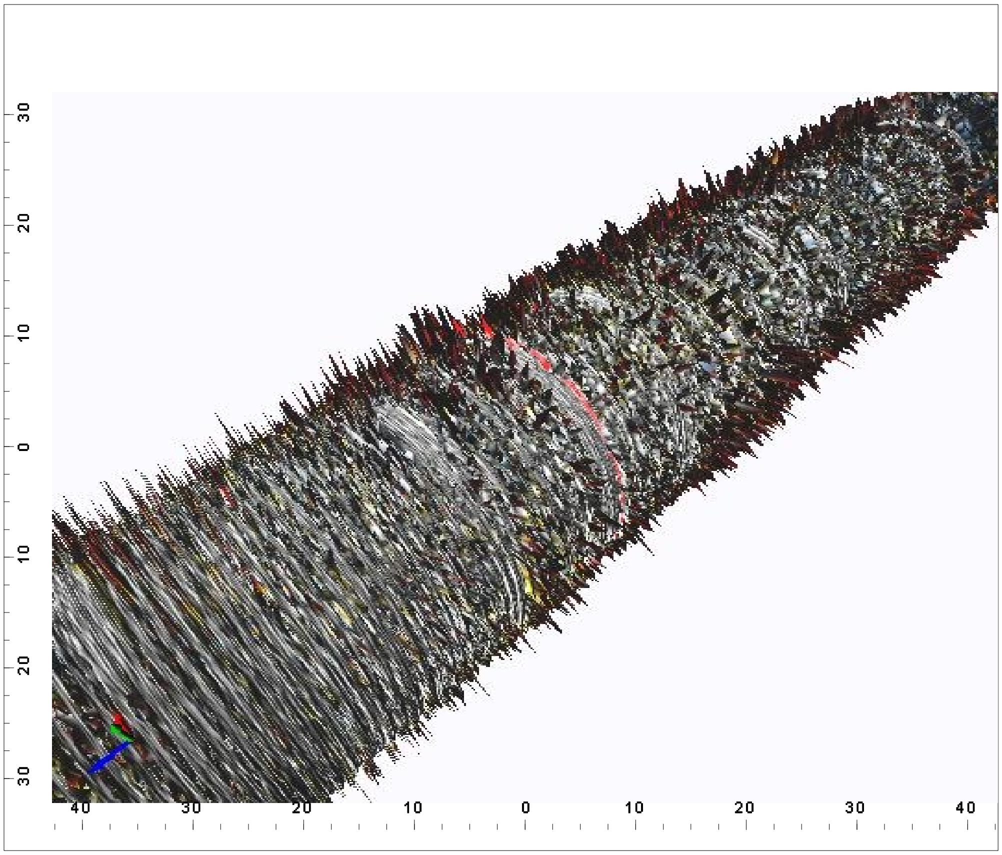

Racking- pipe losing its stability




| r | | ø |
|---|---|---|
| 10.0% | 1.8in | 5.0% |
| 6.7% | 1.2in | 3.3% |
| 3.3% | 0.6in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.6in | -1.7% |
| -6.7% | -1.2in | -3.3% |
| -10.0% | -1.8in | -5.0% |

# Flat Profile Report

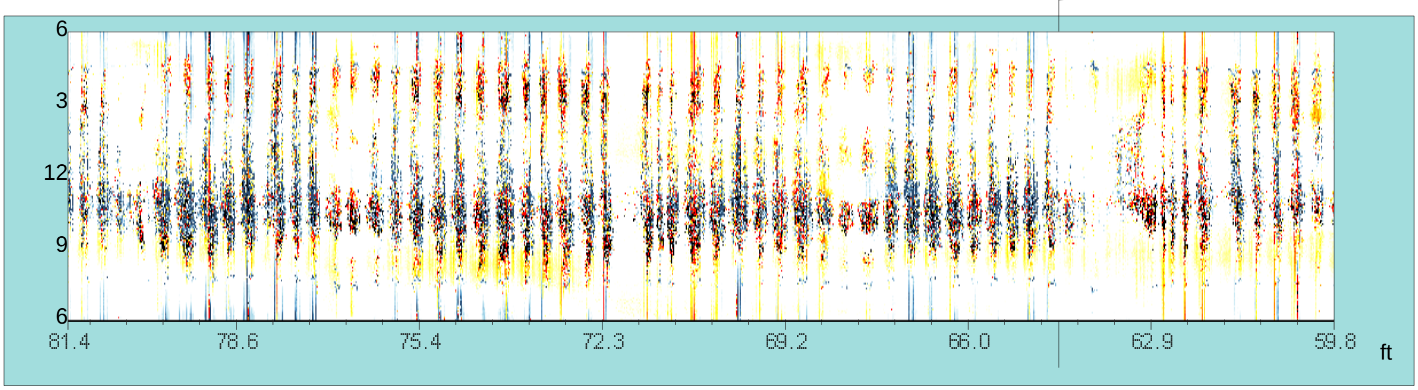
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 222+00HDPE | Date | 8/3/2016 |
| City | Auburn, AL | | | Material | HDPE |
| Start No | 222+00W | Finish No | 222+00E | Pipeline Length | 130 ft |
| Location | Downstream | Location | Downstream | Internal Diameter | 35.87 in |




| r | | ø |
|---|---|---|
| 10.0% | 1.8in | 5.0% |
| 6.7% | 1.2in | 3.3% |
| 3.3% | 0.6in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.6in | -1.7% |
| -6.7% | -1.2in | -3.3% |
| -10.0% | -1.8in | -5.0% |

| | |
|---|---|
| Horizontal 36.3in 1.1% | True 36.1in |
| Vertical 35.6in -0.8% | Ovality 2.2% |
| Capacity $0.34in^2$ 0.0% | |
| Max 36.2in 1.0% | |
| Min 35.0in -2.3% | |

ft 64ft



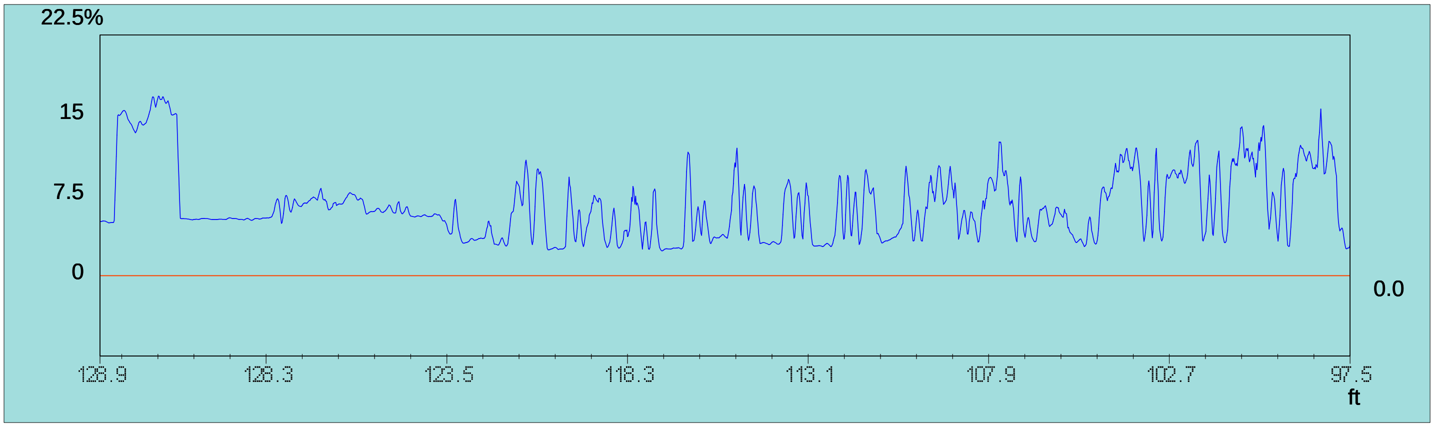

Comments

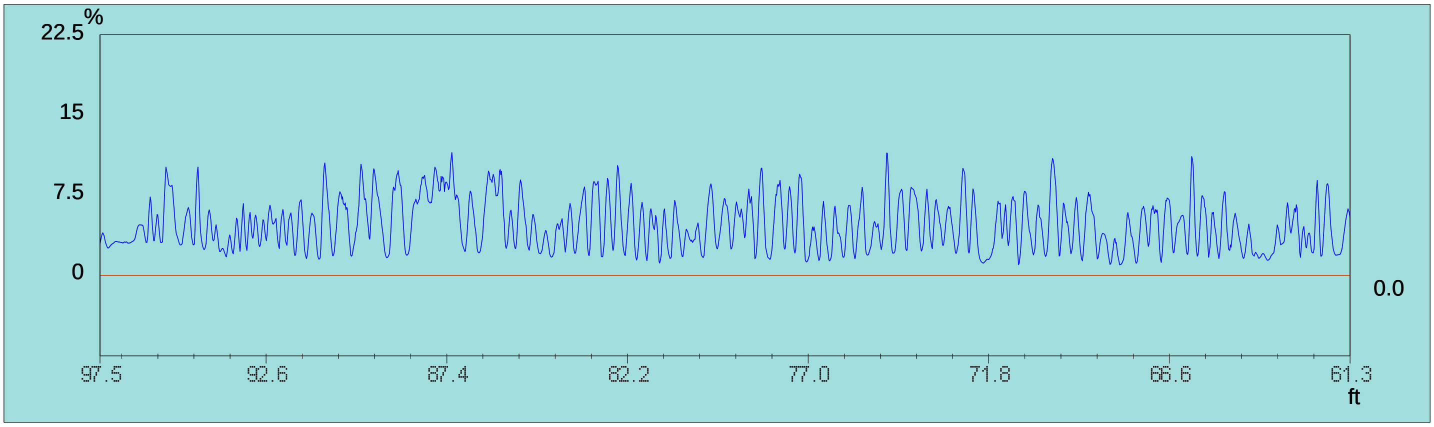
# Ovality Analysis Report

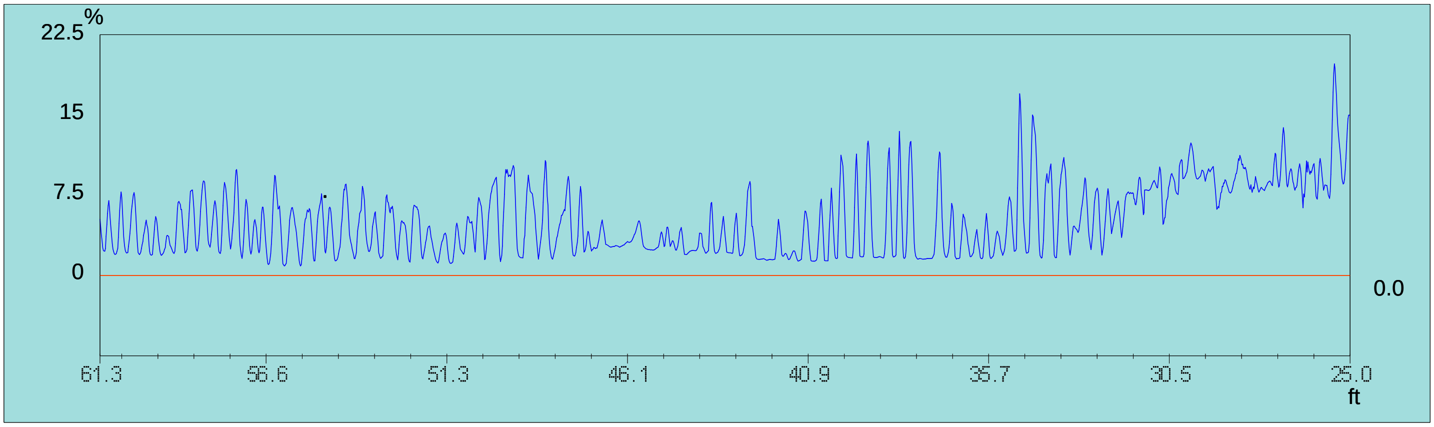
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 222+00HDPE | Date | 8/3/2016 |
| City | Auburn, AL | | | Material | HDPE |
| Start No | 222+00W | Finish No | 222+00E | Pipeline Length | 130ft |
| Location | Downstream | Location | Downstream | Internal Diameter | 35.87in |

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

90% - Fractile: 9.5%, Exceeded limits: 100.0%



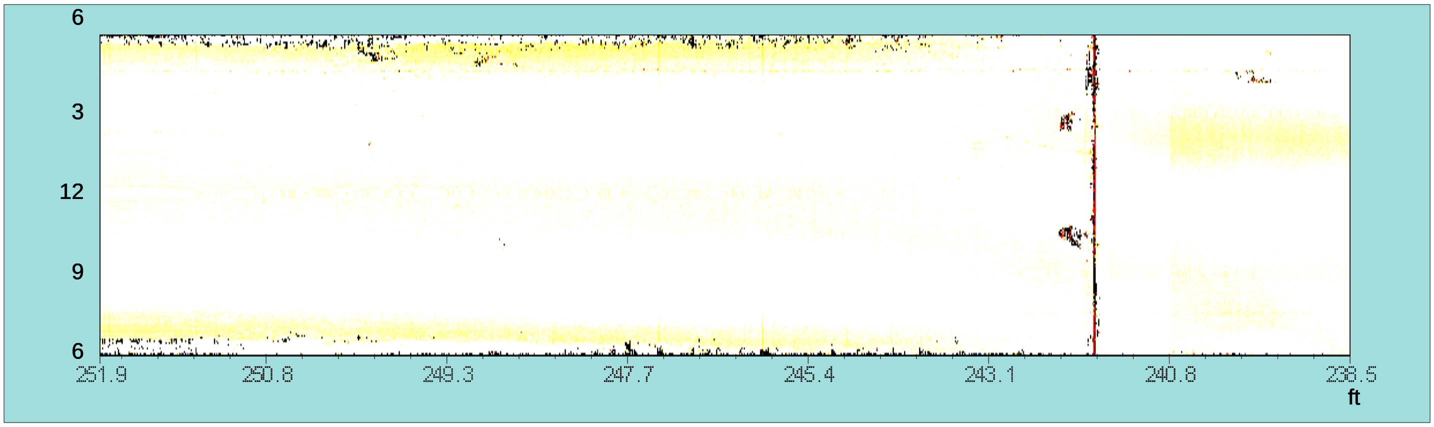



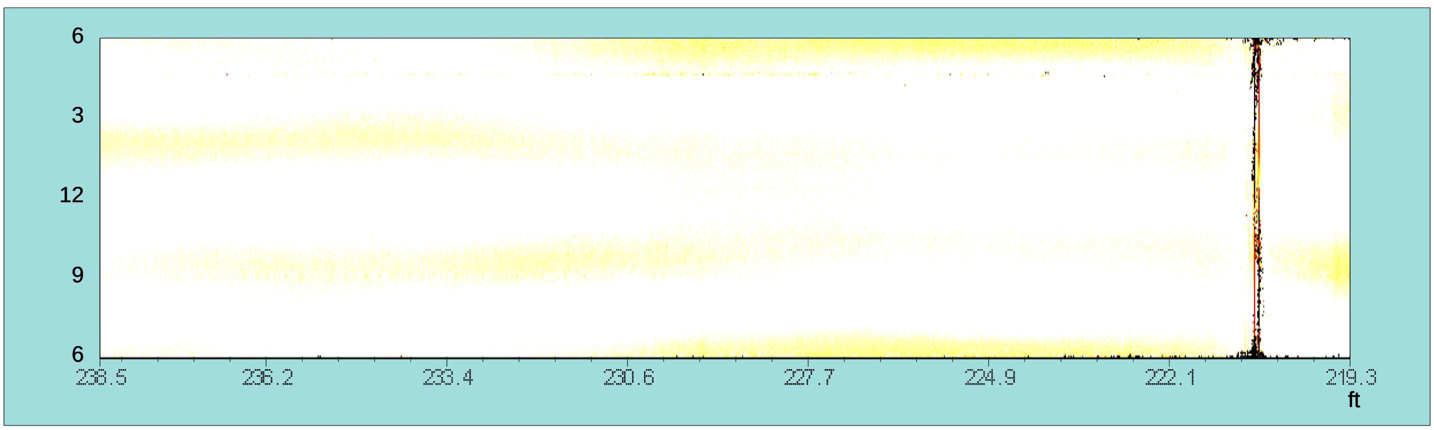

Comments

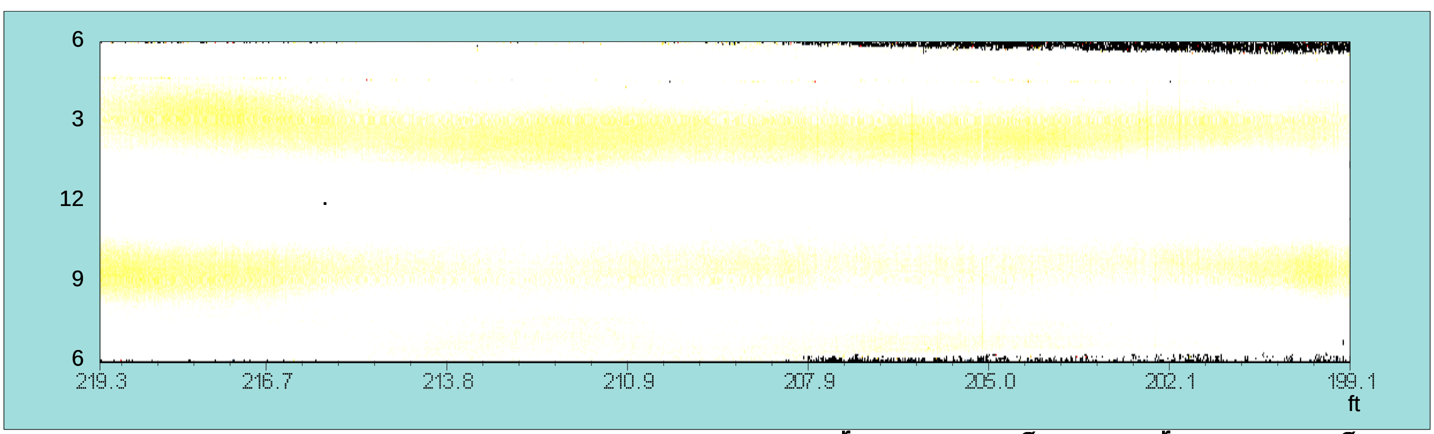
r
10.
6.7
3.3
0%

# Flat Analysis Report

| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 222+00 | Date | 8/3/2016 |
| City | Auburn, AL | | | Material | HDPE/PVC |
| Start No | 222+00W | Finish No | 222+00E | Pipeline Length | 254.9ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87in |



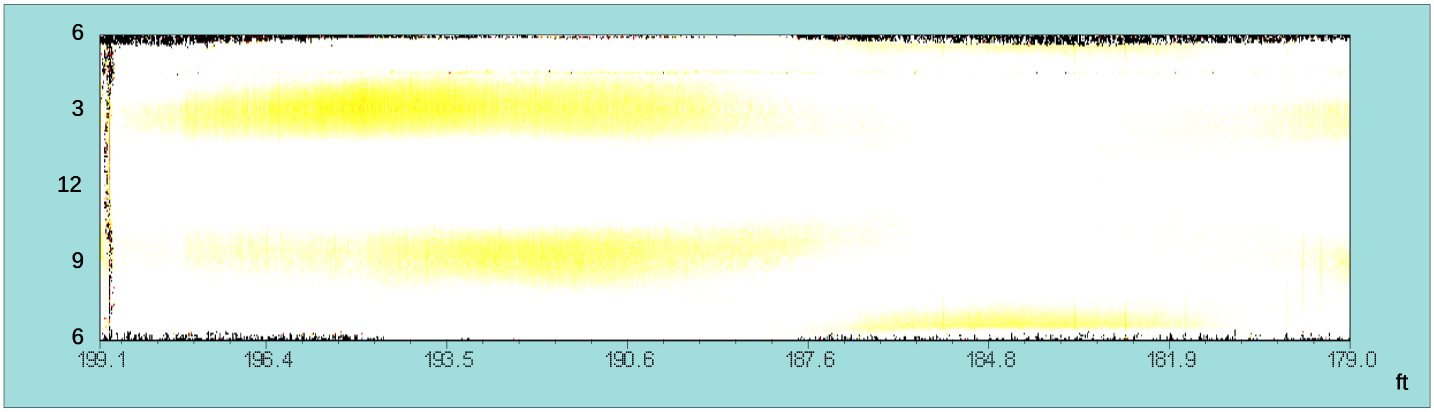



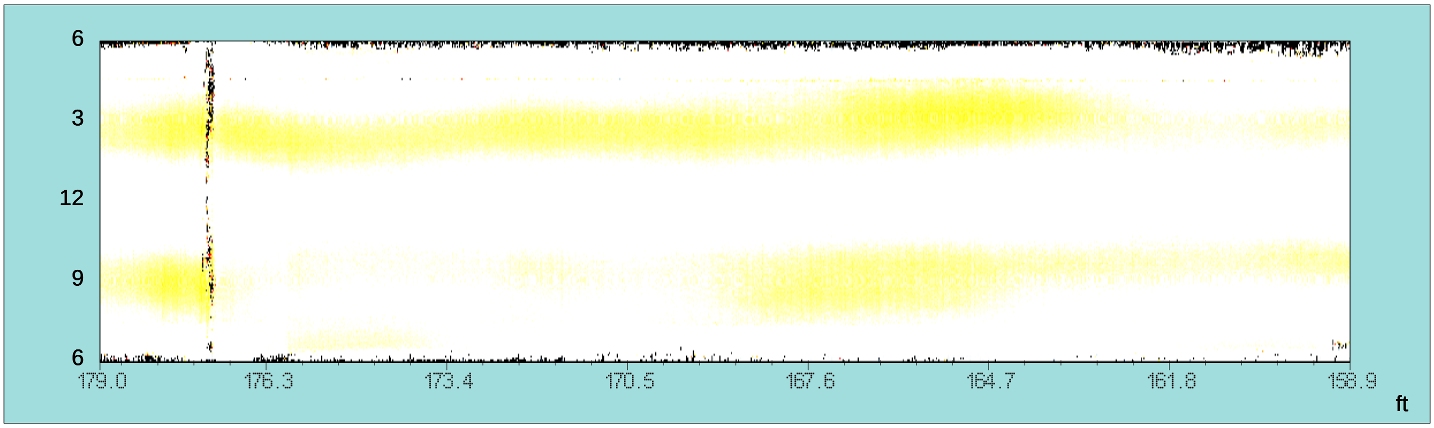



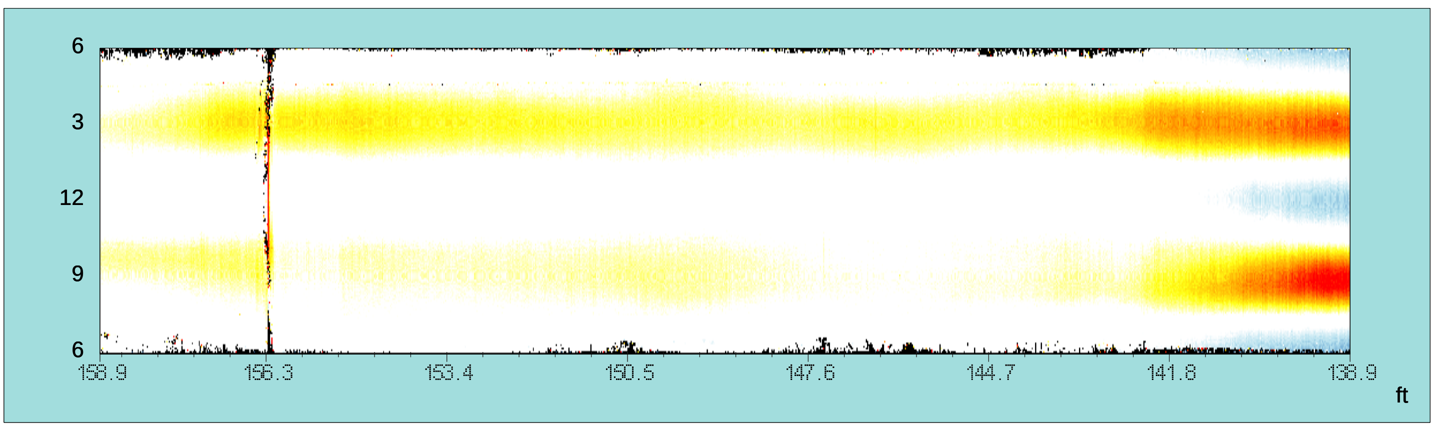

Comments

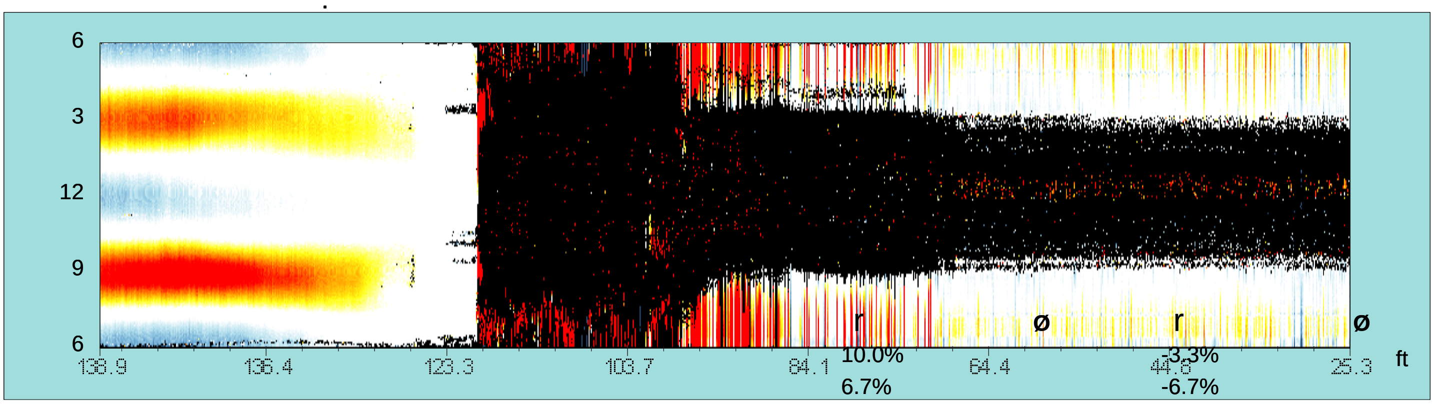
| | r | | ø | | r | | ø |
|---|---|---|---|---|---|---|---|
| red | 10.0% | 1.8in | 5.0% | light blue | -3.3% | -0.6in | -1.7% |
| orange | 6.7% | 1.2in | 3.3% | blue | -6.7% | -1.2in | -3.3% |
| yellow | 3.3% | 0.6in | 1.7% | dark blue | -10.0% | -1.8in | -5.0% |
| white | 0% | 0.0in | 0% | | | | |

# Flat Analysis Report



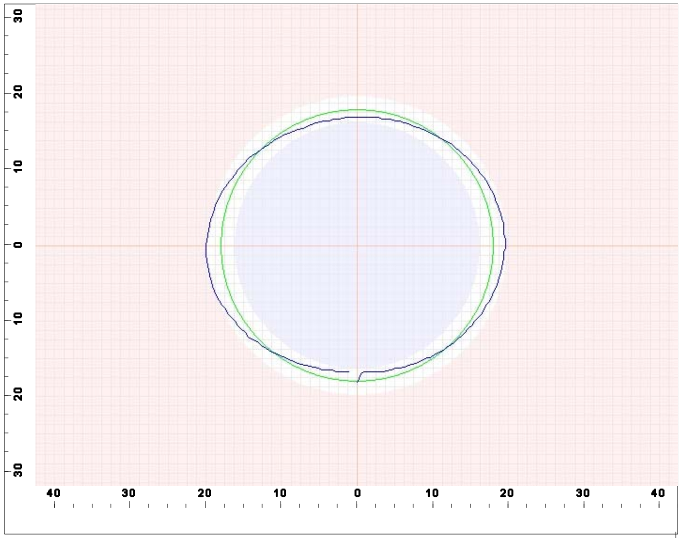



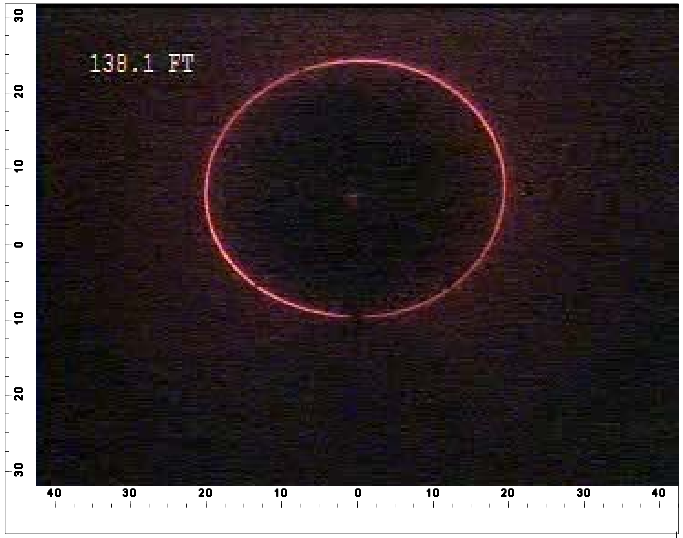



# Flat Observations Report

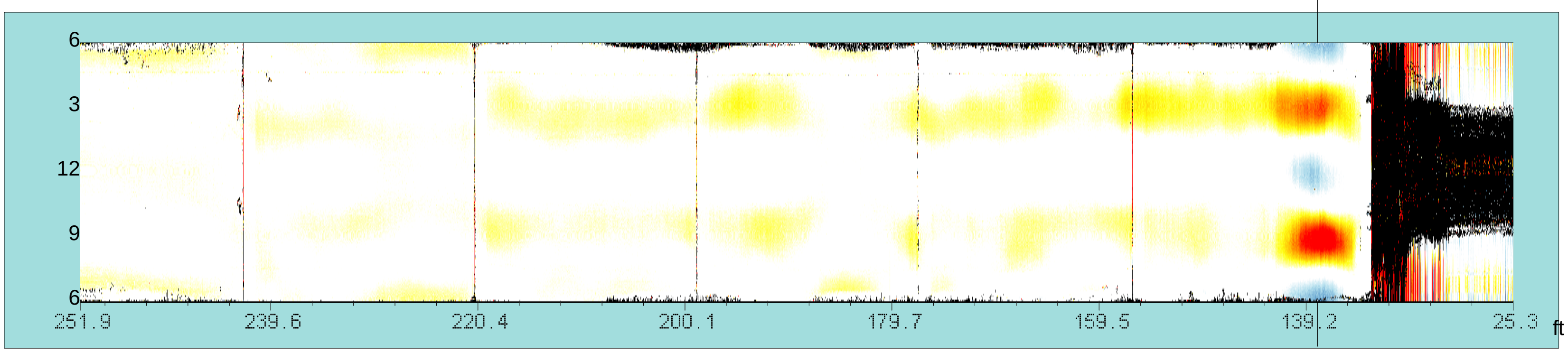
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 222+00 | Date | 8/3/2016 |
| City | Auburn, AL | | | Material | HDPE/PVC |
| Start No | 222+00W | Finish No | 222+00E | Pipeline Length | 254.9 ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87 in |




8.0% Overall Ovality Deformation




The start of the PVC pipe from the junction box




| r | | ø |
|---|---|---|
| 10.0% | 1.8in | 5.0% |
| 6.7% | 1.2in | 3.3% |
| 3.3% | 0.6in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.6in | -1.7% |
| -6.7% | -1.2in | -3.3% |
| -10.0% | -1.8in | -5.0% |


CleanFlow Systems
Ph: +64 9 4799901
www.cleanflowsystems.com

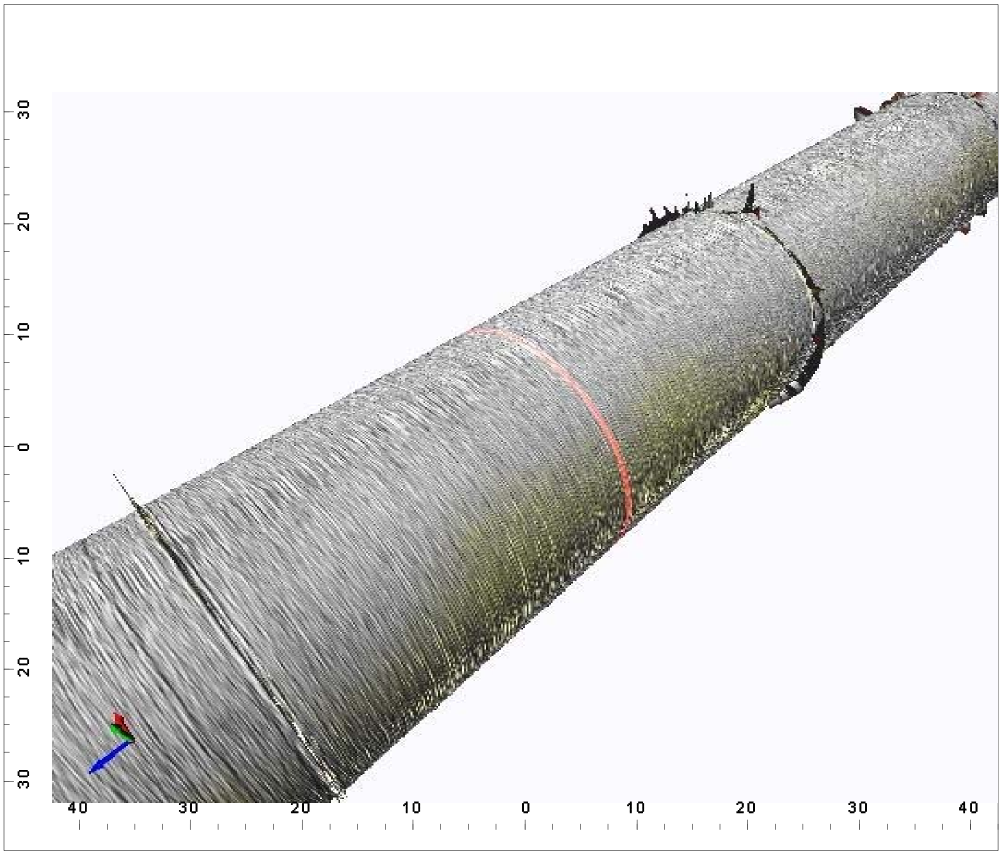





Printed: 8/9/2016

# Flat Profile Report

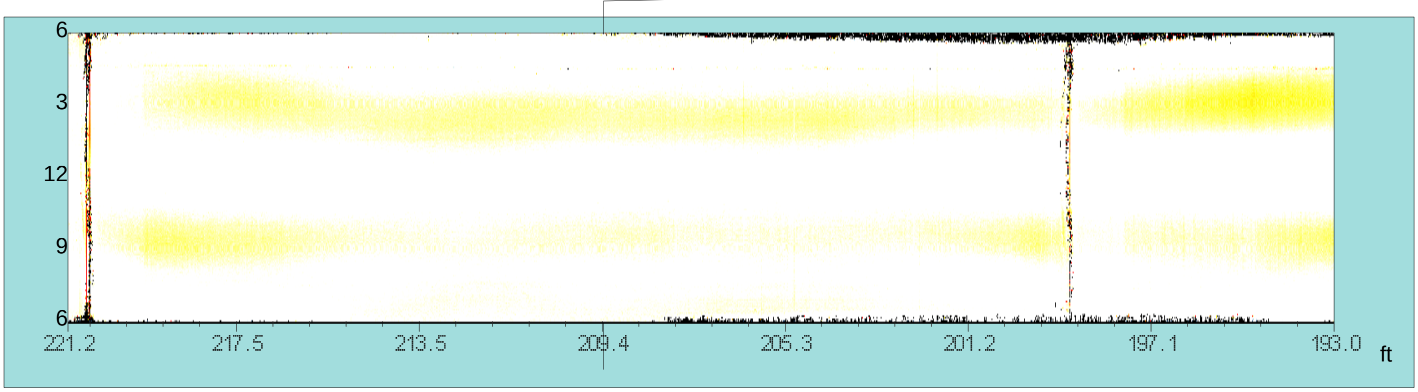
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 222+00 | Date | 8/3/2016 |
| City | Auburn, AL | | | Material | HDPE/PVC |
| Start No | 222+00W | Finish No | 222+00E | Pipeline Length | 254.9 ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87 in |




| r | | ø |
|---|---|---|
| 10.0% | 1.8in | 5.0% |
| 6.7% | 1.2in | 3.3% |
| 3.3% | 0.6in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.6in | -1.7% |
| -6.7% | -1.2in | -3.3% |
| -10.0% | -1.8in | -5.0% |

| | |
|---|---|
| Horizontal 36.9in 2.9% | True 36.6in |
| Vertical 36.6in 2.0% | Ovality 0.9% |
| Capacity $43.13in^2$ 4.3% | |
| Max 37.0in 3.2% | |
| Min 36.3in 1.3% | |

ft 209ft



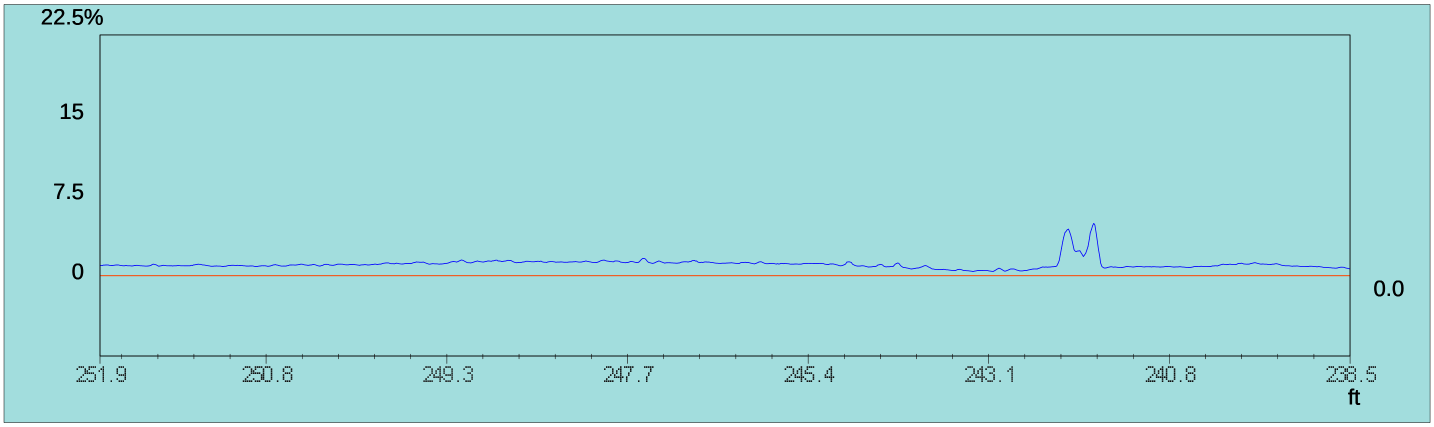

Comments

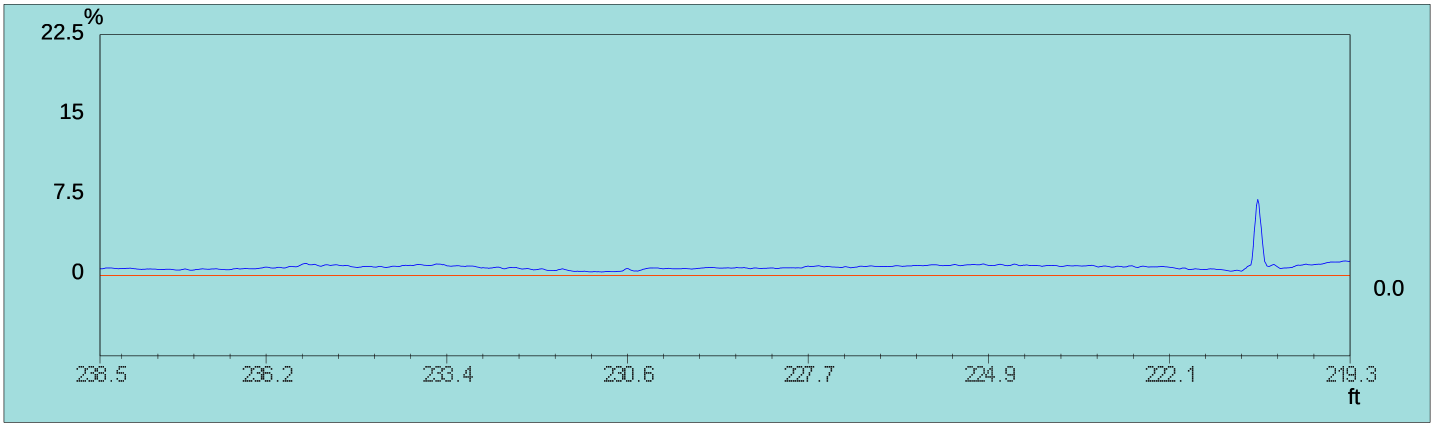
# Ovality Analysis Report

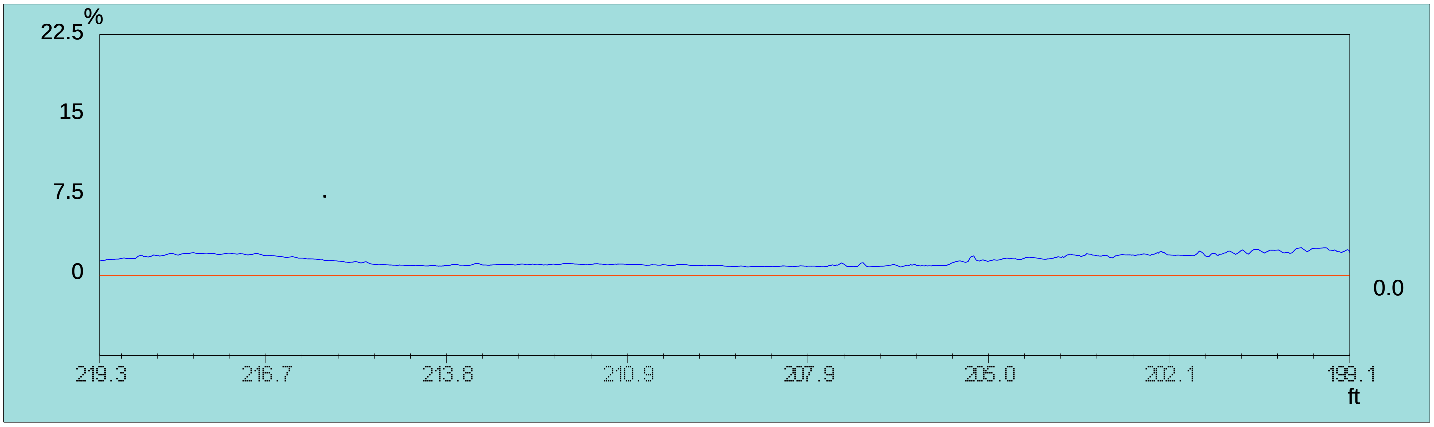
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 222+00 | Date | 8/3/2016 |
| City | Auburn, AL | | | Material | HDPE/PVC |
| Start No | 222+00W | Finish No | 222+00E | Pipeline Length | 254.9ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87in |

90% - Fractile: 3.8%, Exceeded limits: 99.4%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



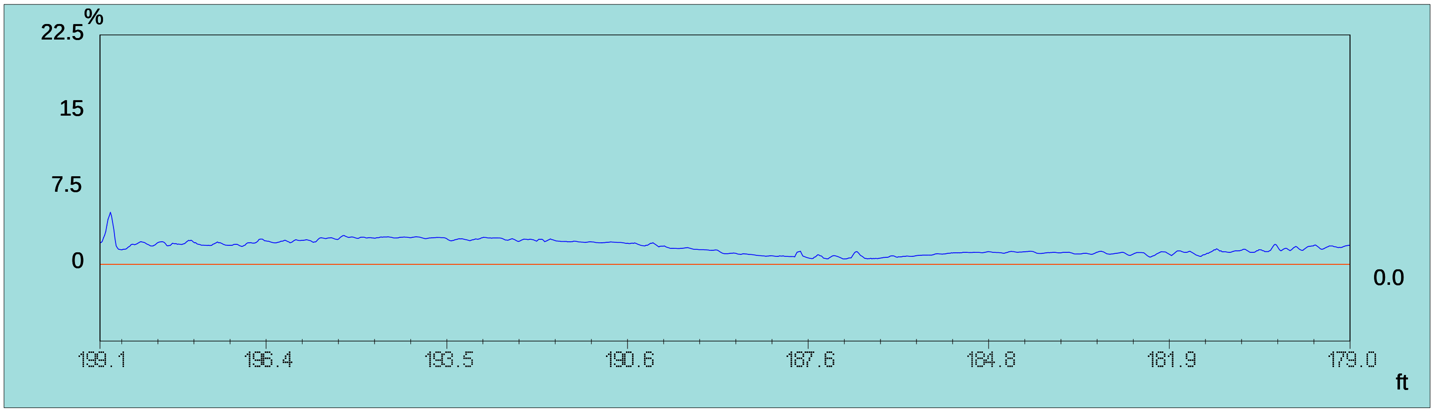



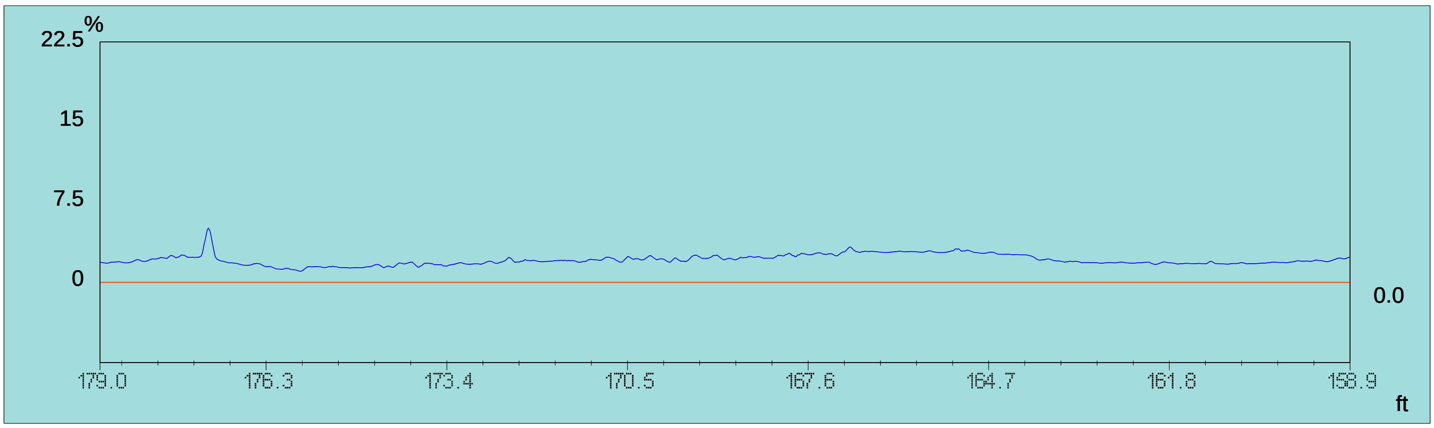



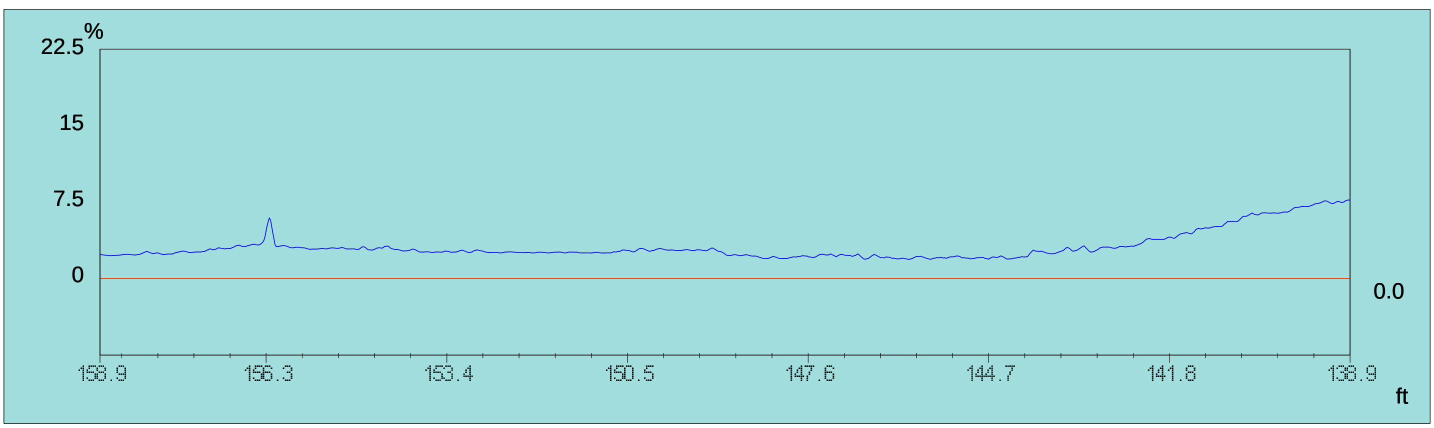

Comments

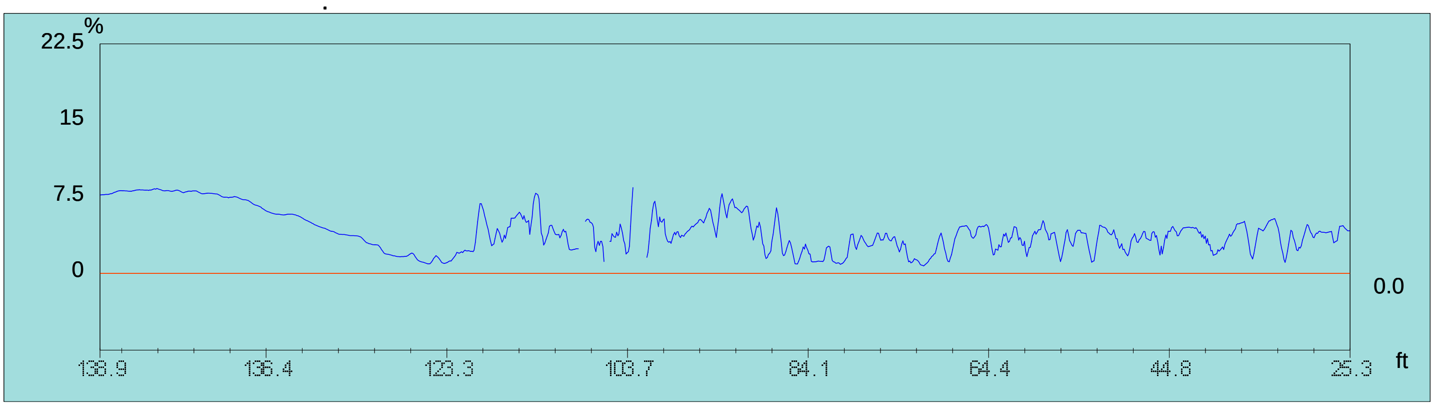
r
10.
6.7
3.3
0%

# Ovality Analysis Report



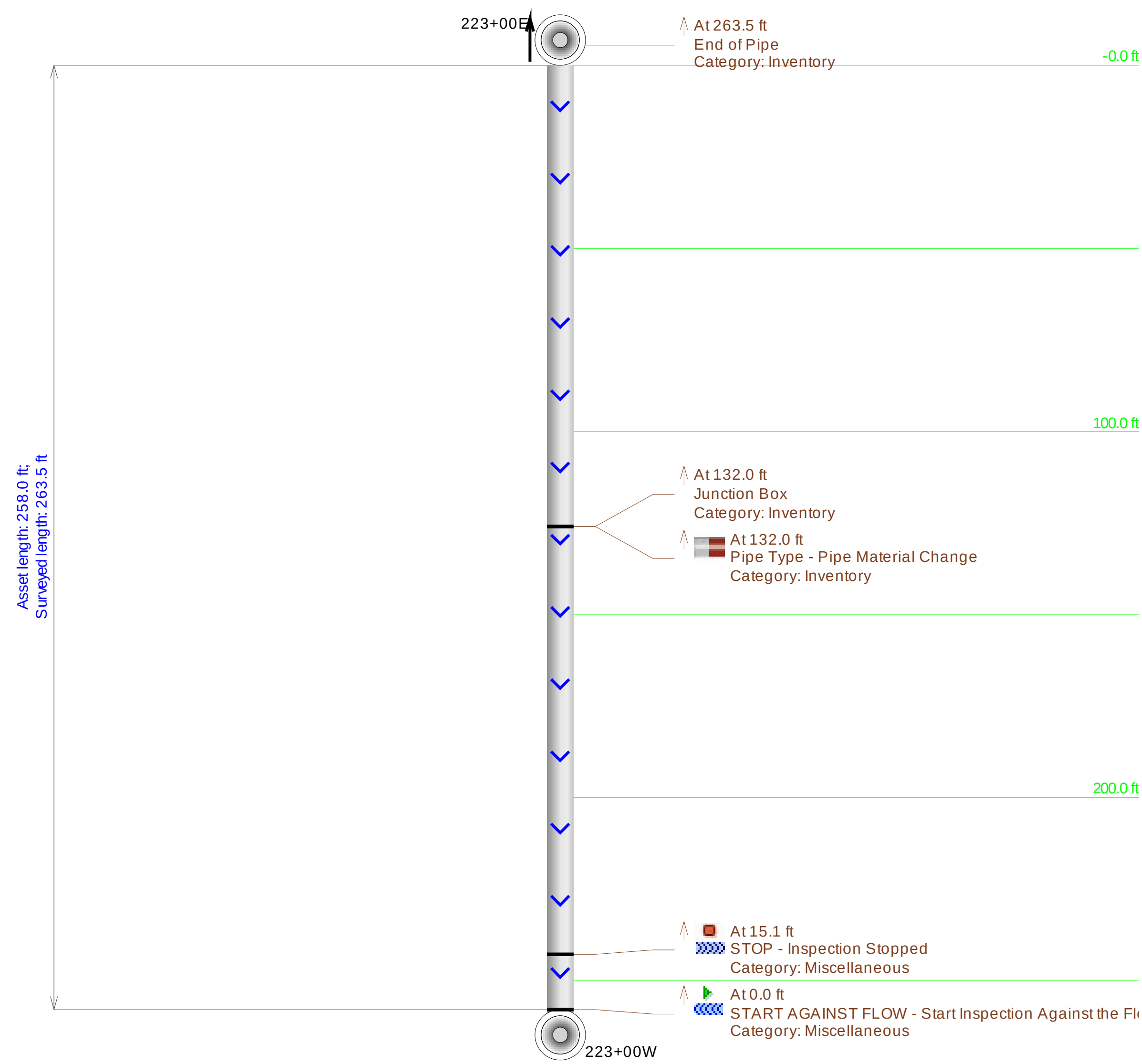
AET Services Robotic Inspections
623 Rices Creek Road
Liberty, SC 29657
1-800-990-8406

# TV Inspection with Pipe-Run Graph

| Project Name: | Mainline ID: | City: | Address: |
|---|---|---|---|
| Auburn University Research | 223+00W-E | Auburn Al | Cox Rd |

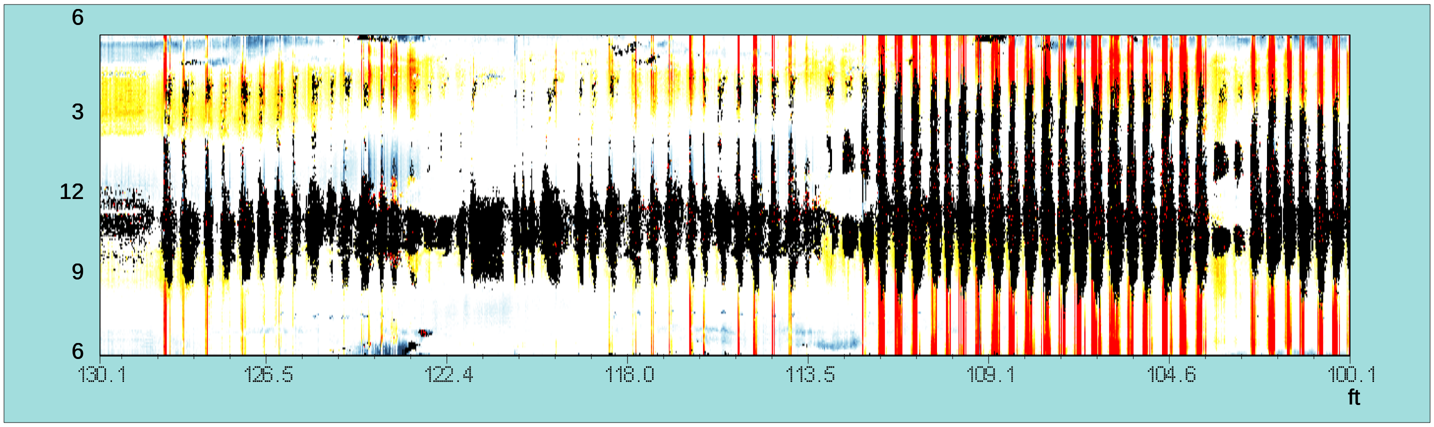
| Start date/time: | Pipe width: | Pipe height: | Pipe type: | Surface condition: |
|---|---|---|---|---|
| 7/28/2016 | 36 | 36 | Polyethylene | Asphalt |

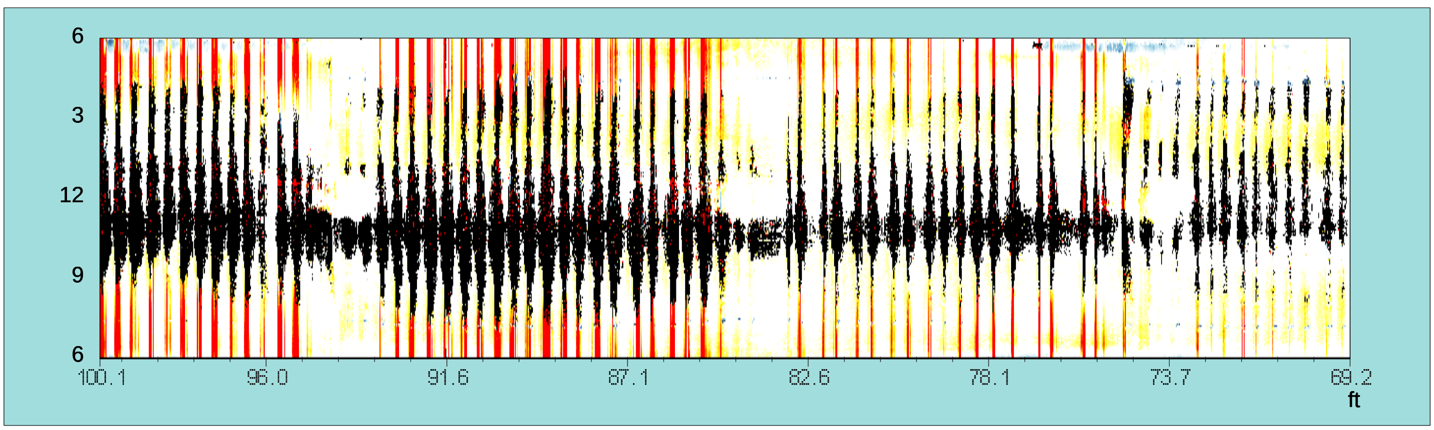
| Direction: | Surveyed footage: | Weather: | MediaLabel |
|---|---|---|---|
| Upstream | 263.5 | Dry | |

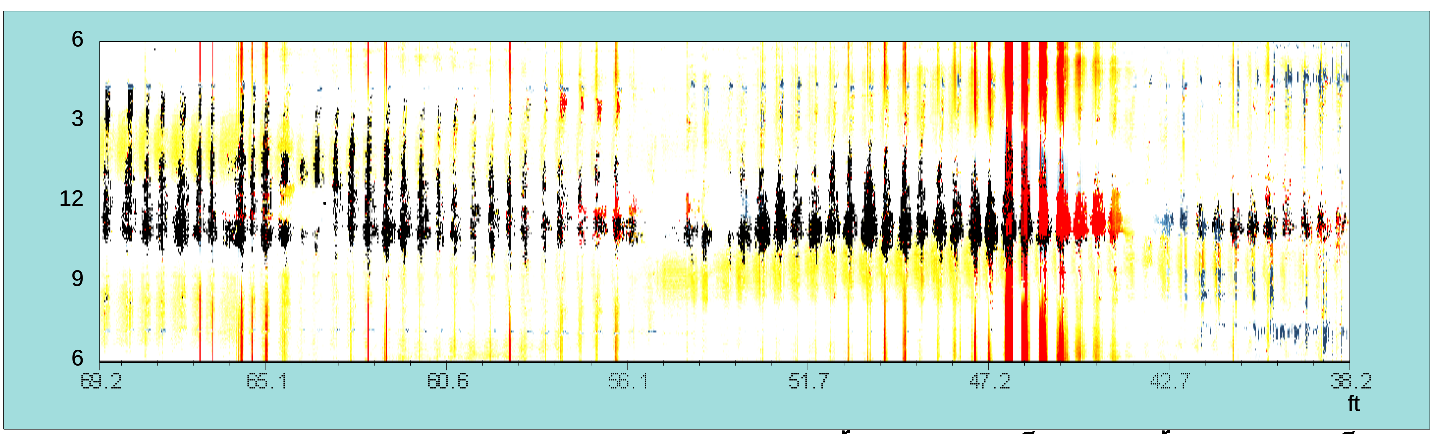


# Flat Analysis Report

| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 223+00HDPE | Date | 8/8/2016 |
| City | Auburn, AL | | | Material | HDPE |
| Start No | 223+00W | Finish No | 223+00E | Pipeline Length | 132ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87in |



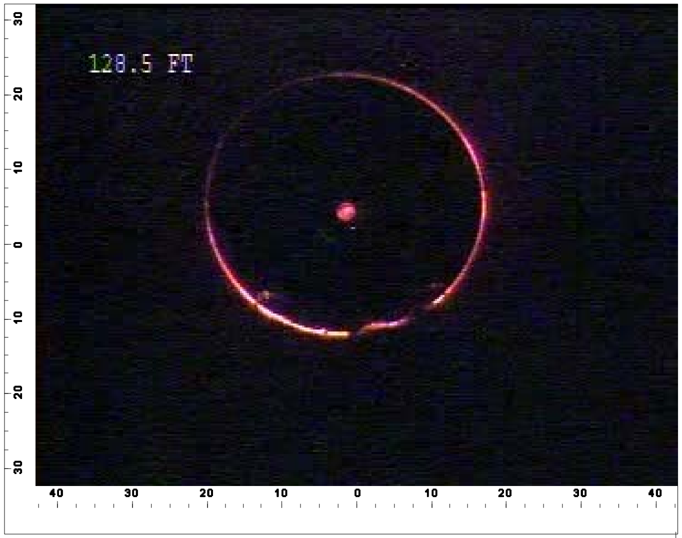




Comments

| r | | ø | r | | ø |
|---|---|---|---|---|---|
| 10.0% | 1.8in | 5.0% | -3.3% | -0.6in | -1.7% |
| 6.7% | 1.2in | 3.3% | -6.7% | -1.2in | -3.3% |
| 3.3% | 0.6in | 1.7% | -10.0% | -1.8in | -5.0% |
| 0% | 0.0in | 0% | | | |

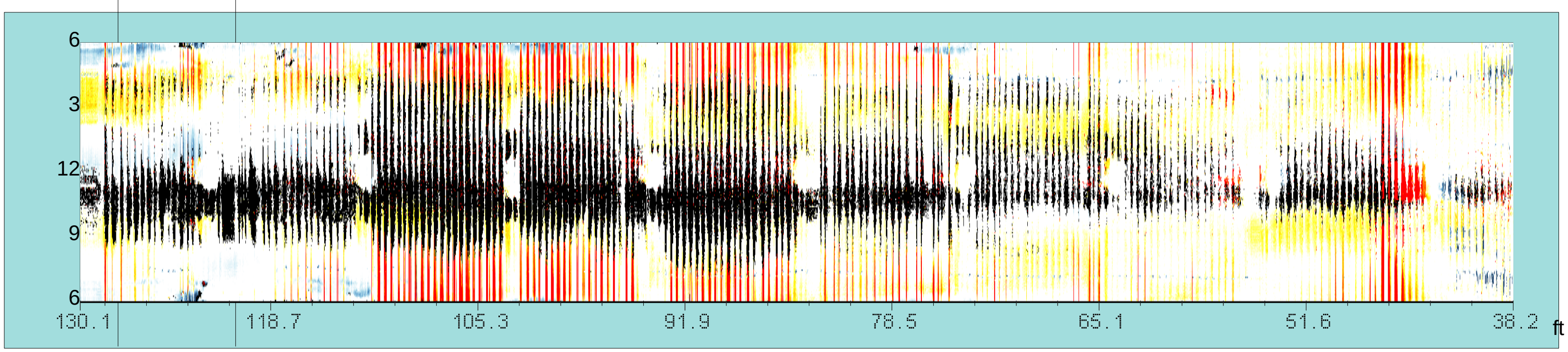
# Flat Observations Report

| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 223+00HDPE | Date | 8/8/2016 |
| City | Auburn, AL | | | Material | HDPE |
| Start No | 223+00W | Finish No | 223+00E | Pipeline Length | 132 ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87 in |




4.5% Ovality Deformation

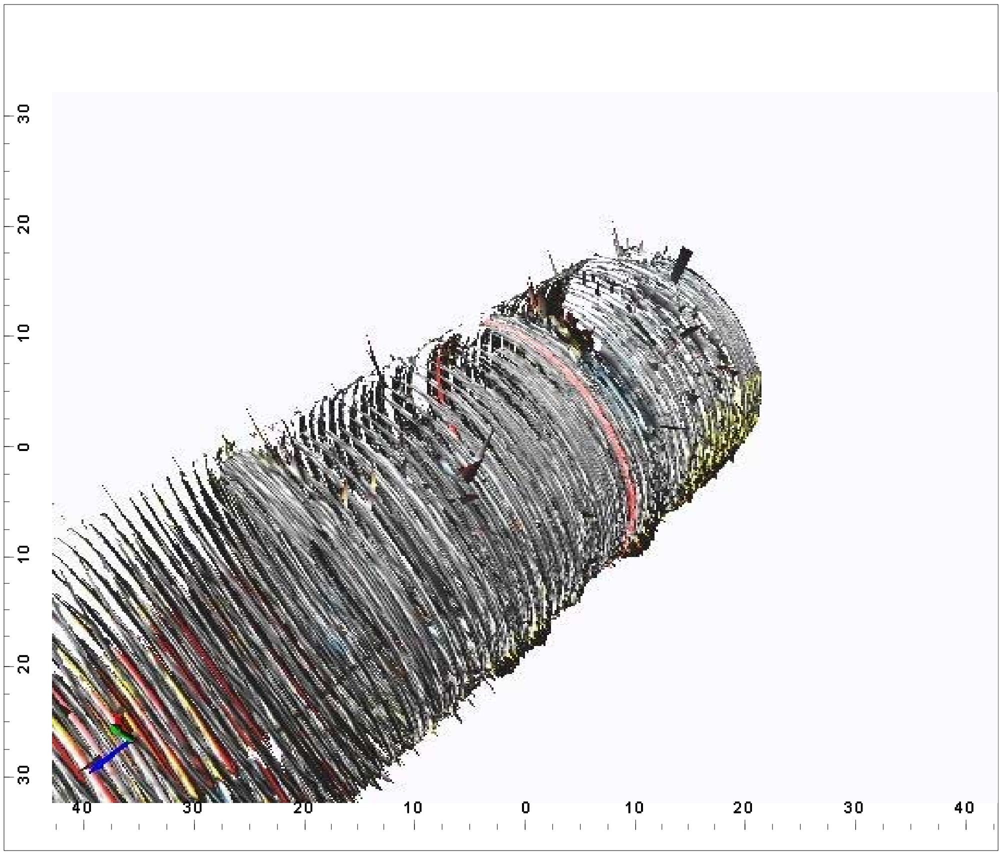

2.0% Ovality Deformation




| r | | ø |
|---|---|---|
| 10.0% | 1.8in | 5.0% |
| 6.7% | 1.2in | 3.3% |
| 3.3% | 0.6in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.6in | -1.7% |
| -6.7% | -1.2in | -3.3% |
| -10.0% | -1.8in | -5.0% |

# Flat Profile Report

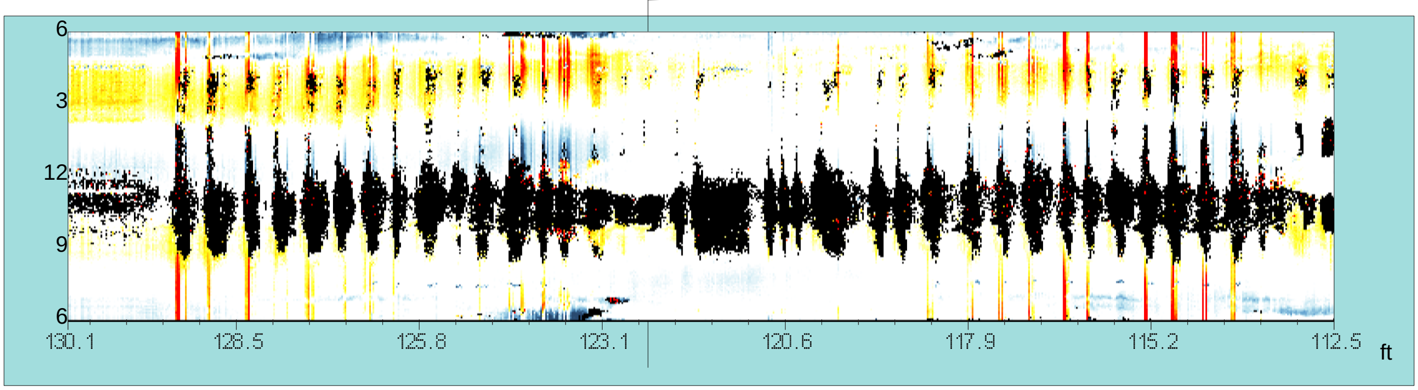
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 223+00HDPE | Date | 8/8/2016 |
| City | Auburn, AL | | | Material | HDPE |
| Start No | 223+00W | Finish No | 223+00E | Pipeline Length | 132 ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87 in |



| r | | ø |
|---|---|---|
| 10.0% | 1.8in | 5.0% |
| 6.7% | 1.2in | 3.3% |
| 3.3% | 0.6in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.6in | -1.7% |
| -6.7% | -1.2in | -3.3% |
| -10.0% | -1.8in | -5.0% |

| | |
|---|---|
| Horizontal 36.2in 0.9% | True 35.9in |
| Vertical 35.8in -0.2% | Ovality 3.1% |
| Capacity 8.58in$^2$ 0.8% | |
| Max 37.1in 3.3% | |
| Min 35.3in -1.6% | |

ft 123ft




Comments

CleanFlow Systems





Ph:+64 9 4799901 www.cleanflowsystems.com

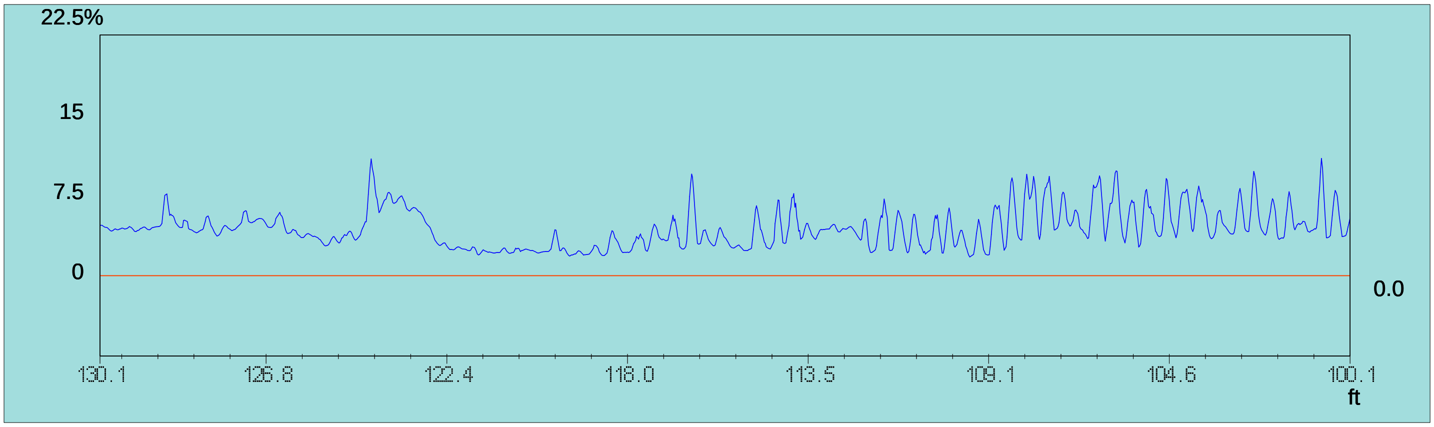
Printed: 8/8/2016

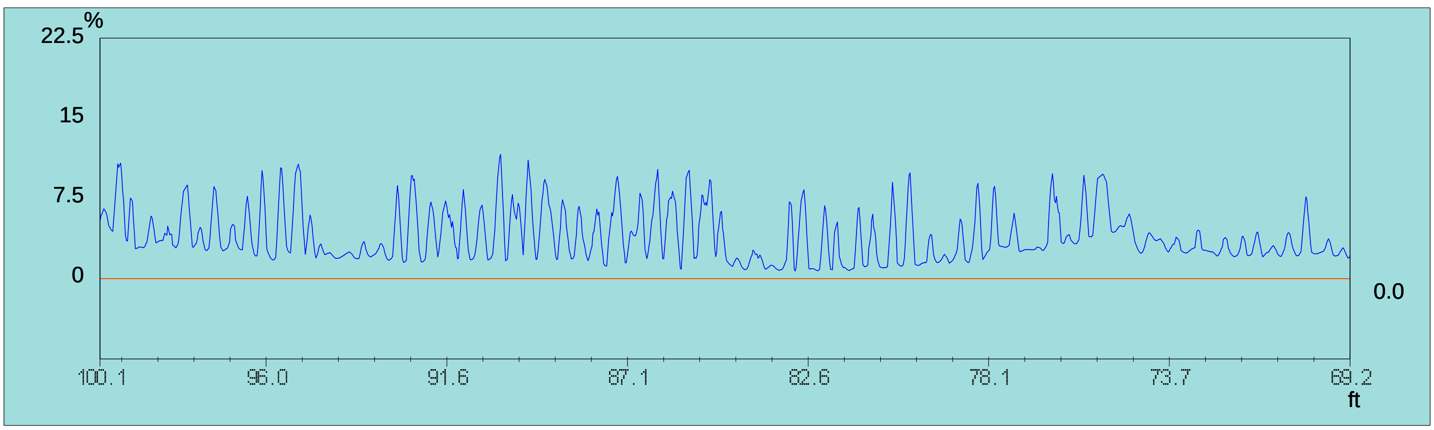
# Ovality Analysis Report

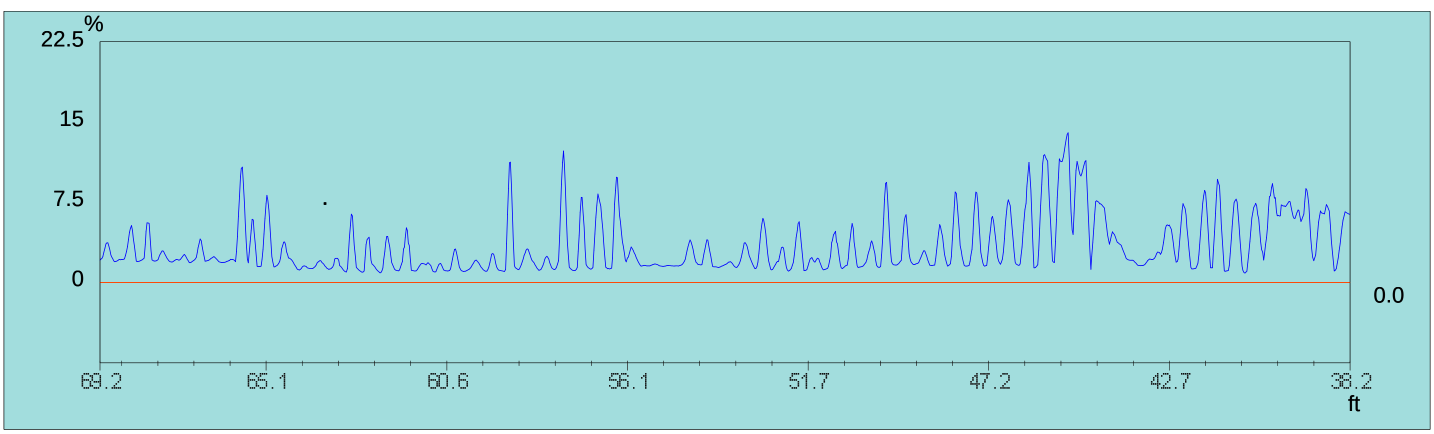
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 223+00HDPE | Date | 8/8/2016 |
| City | Auburn, AL | | | Material | HDPE |
| Start No | 223+00W | Finish No | 223+00E | Pipeline Length | 132ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87in |

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

90% - Fractile: 7.3%, Exceeded limits: 100.0%



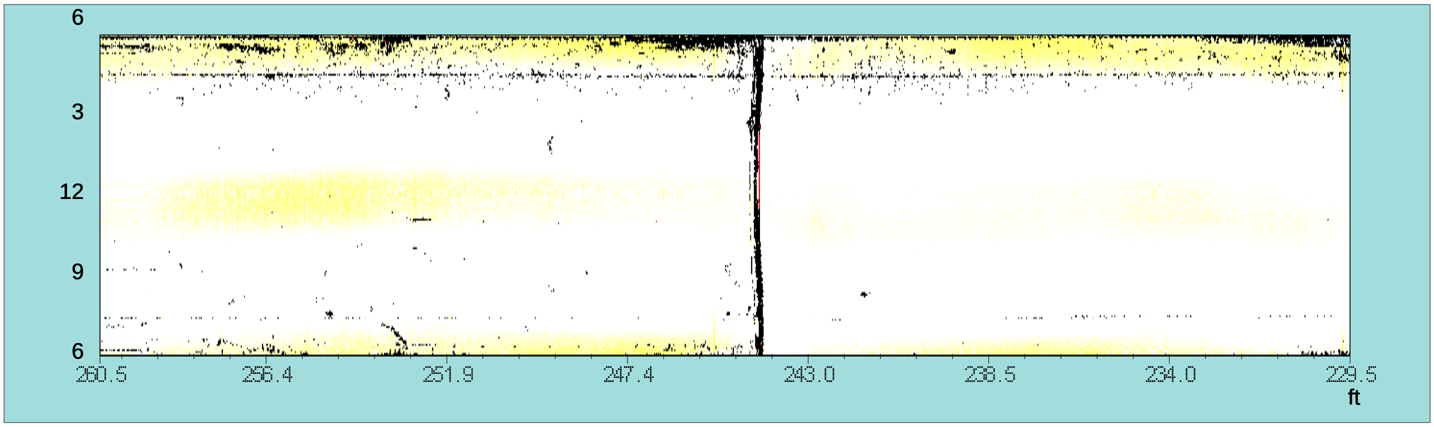



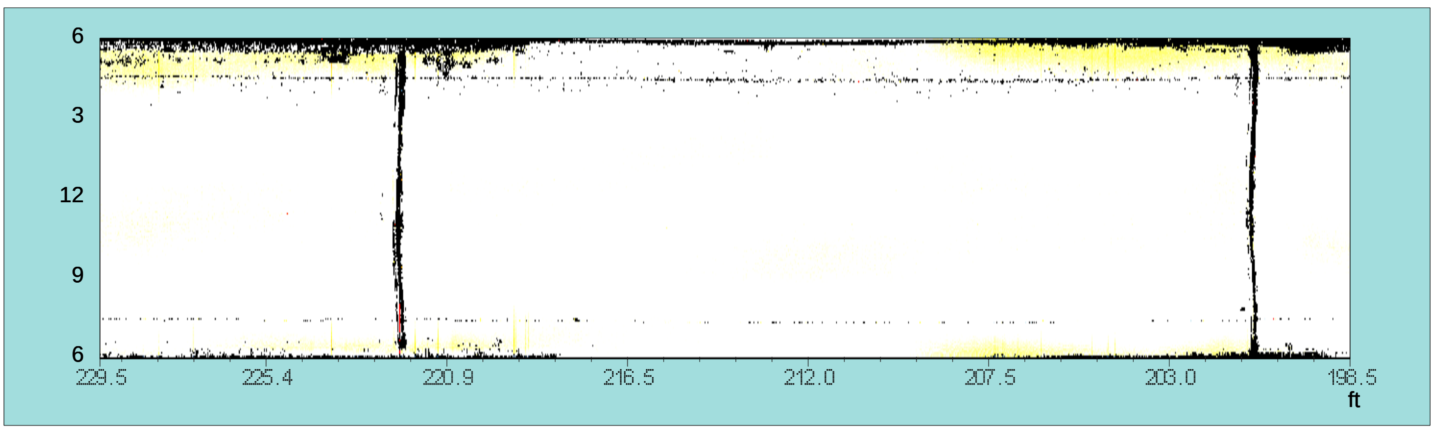

Comments

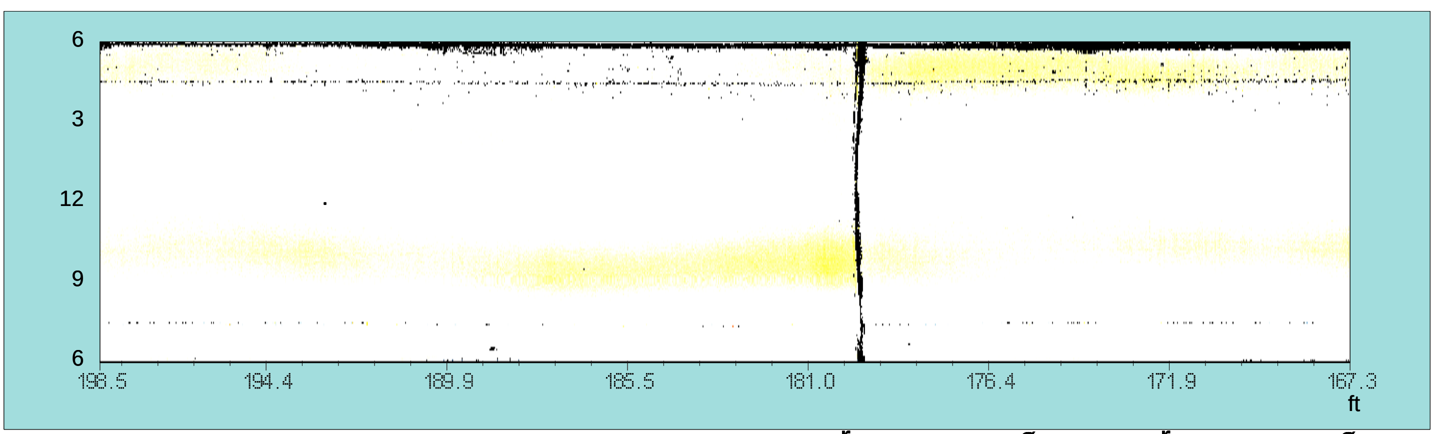
r
10.(
6.7
3.3
0%

# Flat Analysis Report

| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 223+00PVC | Date | 8/8/2016 |
| City | Auburn, AL | | | Material | PVC |
| Start No | 223+00W | Finish No | 223+00E | Pipeline Length | 127ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87in |



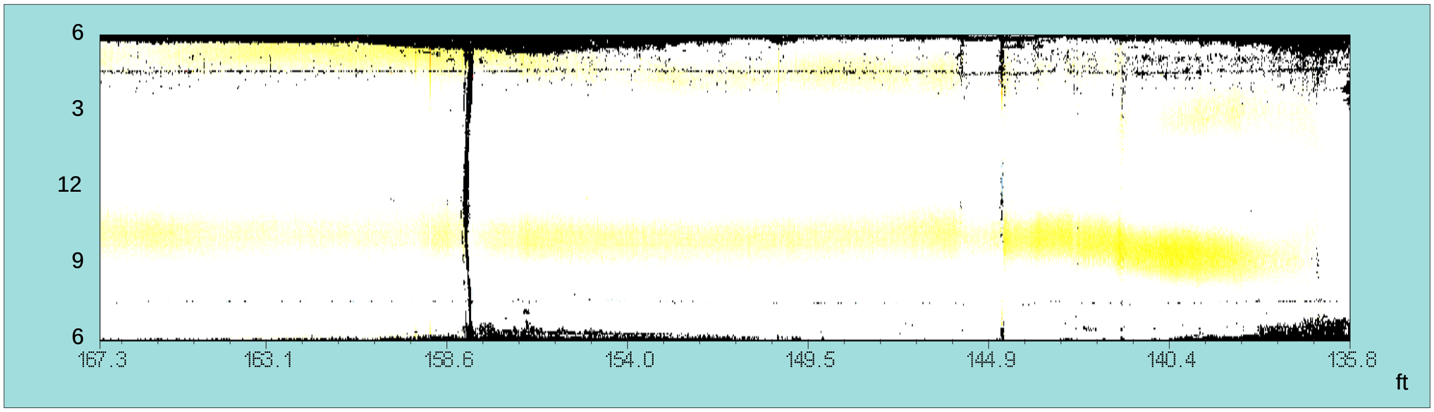




Comments

136-263.5

| r | | ø | r | | ø |
|---|---|---|---|---|---|
| 10.0% | 1.8in | 5.0% | -3.3% | -0.6in | -1.7% |
| 6.7% | 1.2in | 3.3% | -6.7% | -1.2in | -3.3% |
| 3.3% | 0.6in | 1.7% | -10.0% | -1.8in | -5.0% |
| 0% | 0.0in | 0% | | | |

# Flat Analysis Report

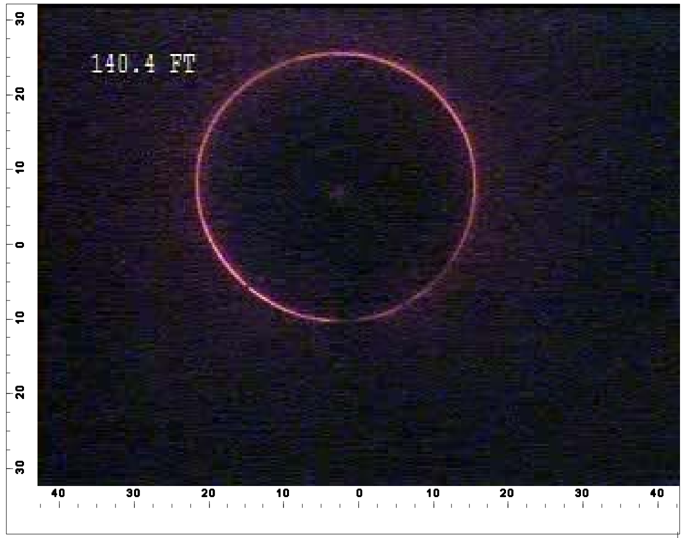



| r | ø | r | ø |
|---|---|---|---|
| 10.0% | | -3.3% | |
| 6.7% | | -6.7% | |
| 3.3% | | -10.0% | |
| 0% | | | |

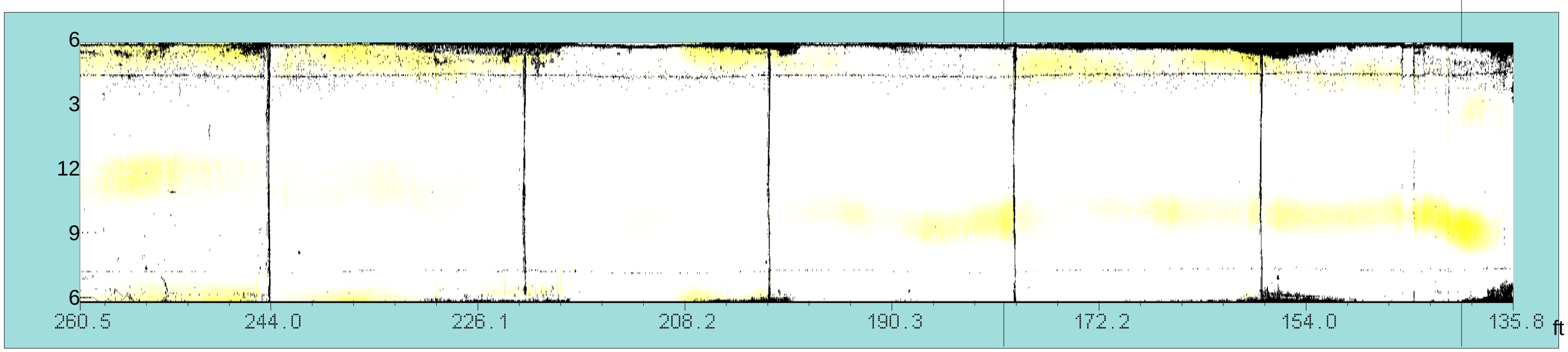
# Flat Observations Report

| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 223+00PVC | Date | 8/8/2016 |
| City | Auburn, AL | | | Material | PVC |
| Start No | 223+00W | Finish No | 223+00E | Pipeline Length | 127 ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87 in |


1.4% Ovality Deformation



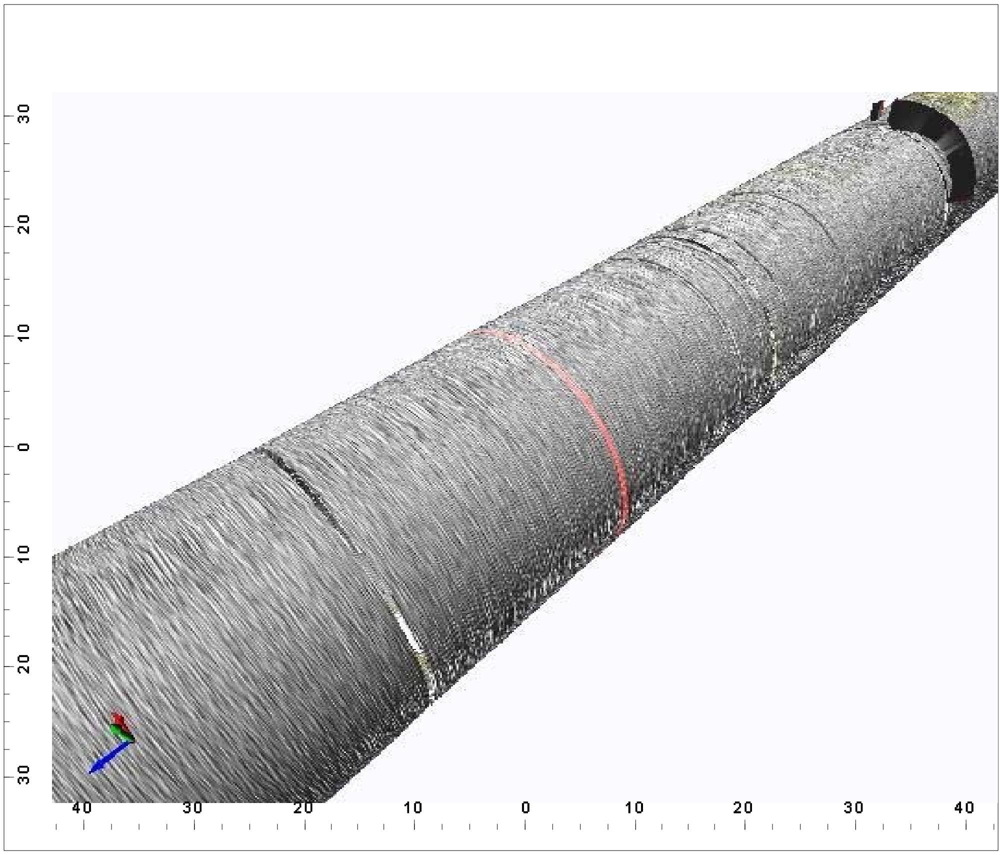

1.7% Overall ovality deformation




| r | | ø |
|---|---|---|
| 10.0% | 1.8in | 5.0% |
| 6.7% | 1.2in | 3.3% |
| 3.3% | 0.6in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.6in | -1.7% |
| -6.7% | -1.2in | -3.3% |
| -10.0% | -1.8in | -5.0% |

# Flat Profile Report

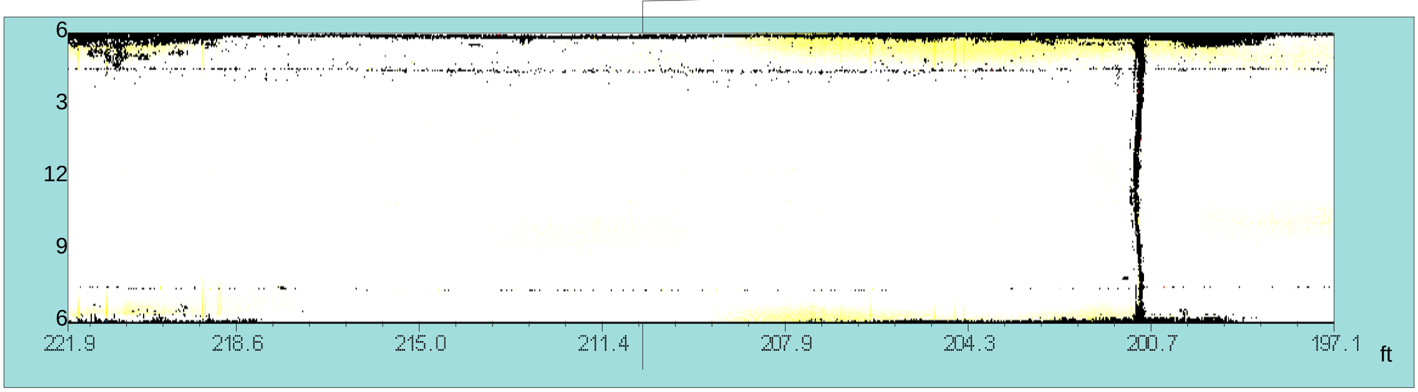
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 223+00PVC | Date | 8/8/2016 |
| City | Auburn, AL | | | Material | PVC |
| Start No | 223+00W | Finish No | 223+00E | Pipeline Length | 127 ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87 in |




| r | | ø |
|---|---|---|
| 10.0% | 1.8in | 5.0% |
| 6.7% | 1.2in | 3.3% |
| 3.3% | 0.6in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.6in | -1.7% |
| -6.7% | -1.2in | -3.3% |
| -10.0% | -1.8in | -5.0% |

| | | | |
|---|---|---|---|
| Horizontal | 36.5in 1.9% | True | 36.5in |
| Vertical | 36.3in 1.3% | Ovality | 0.7% |
| Capacity | 34.08in$^2$ 3.4% | | |
| Max | 36.7in 2.3% | | |
| Min | 36.3in 1.3% | | |

ft 211ft




Comments

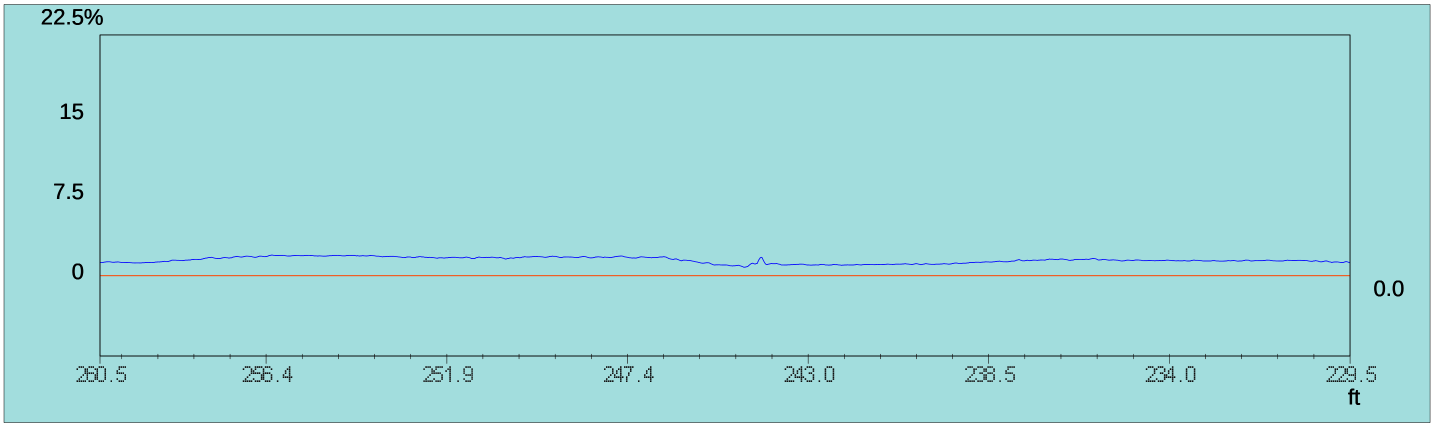
136-263.5

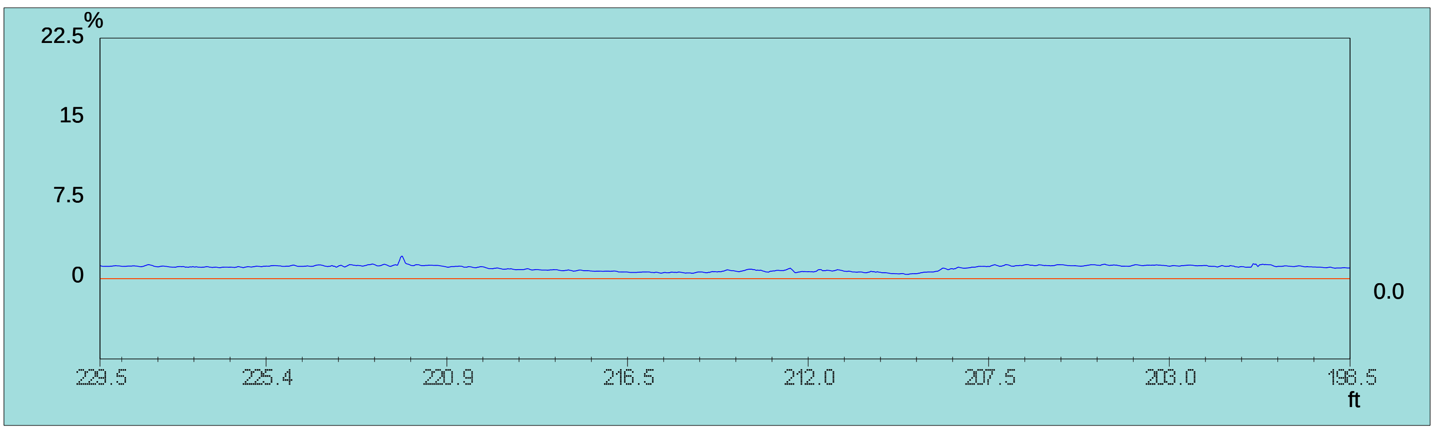
# Ovality Analysis Report

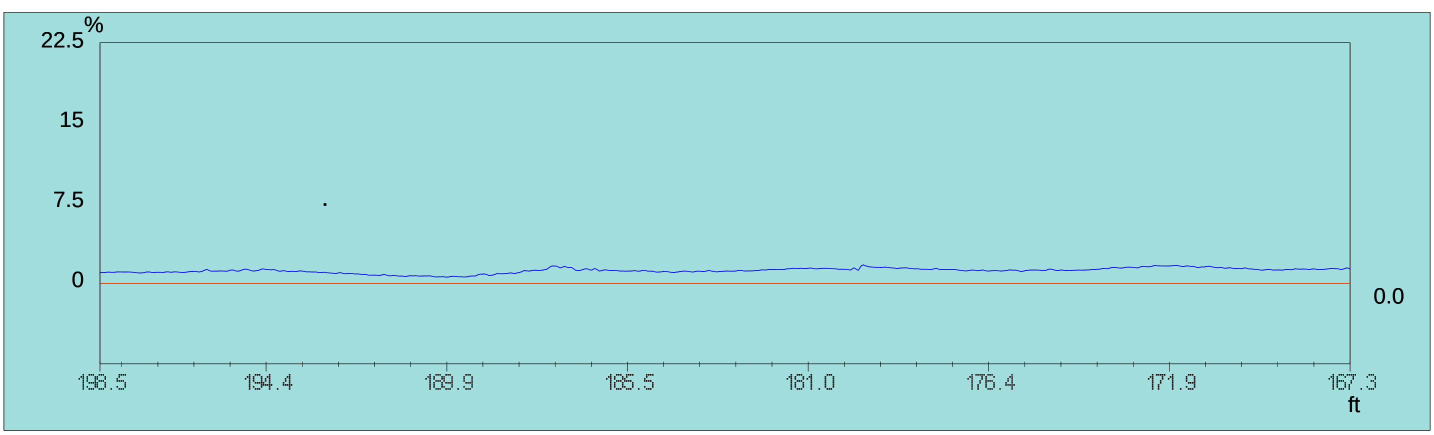
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 223+00PVC | Date | 8/8/2016 |
| City | Auburn, AL | | | Material | PVC |
| Start No | 223+00W | Finish No | 223+00E | Pipeline Length | 127ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87in |

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

90% - Fractile: 1.7%, Exceeded limits: 100.0%



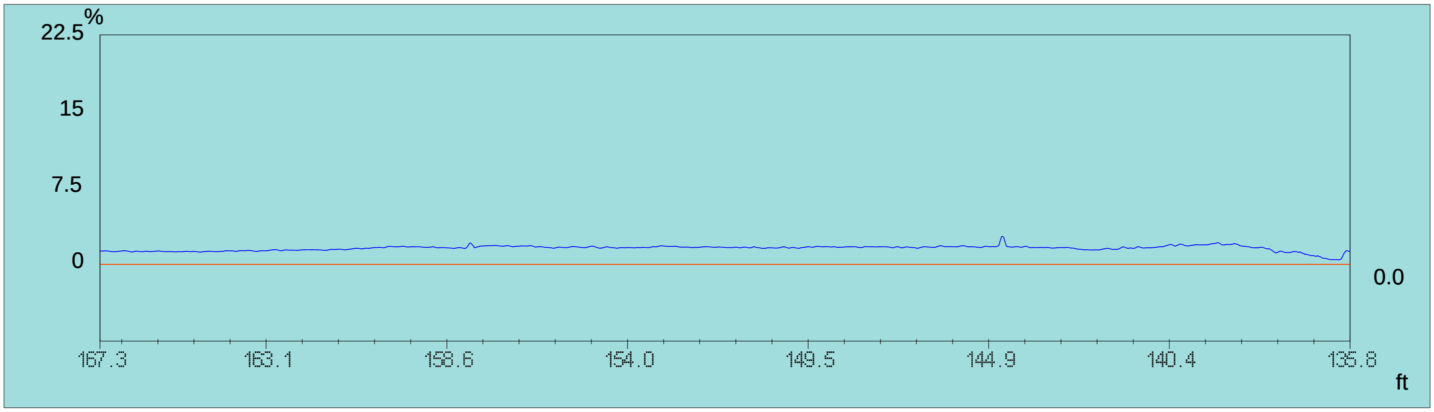



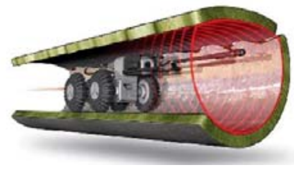

Comments

136-263.5

# Ovality Analysis Report



AET Services  Robotic Inspections
623 Rices Creek Road
Liberty, SC 29657
1-800-990-8406

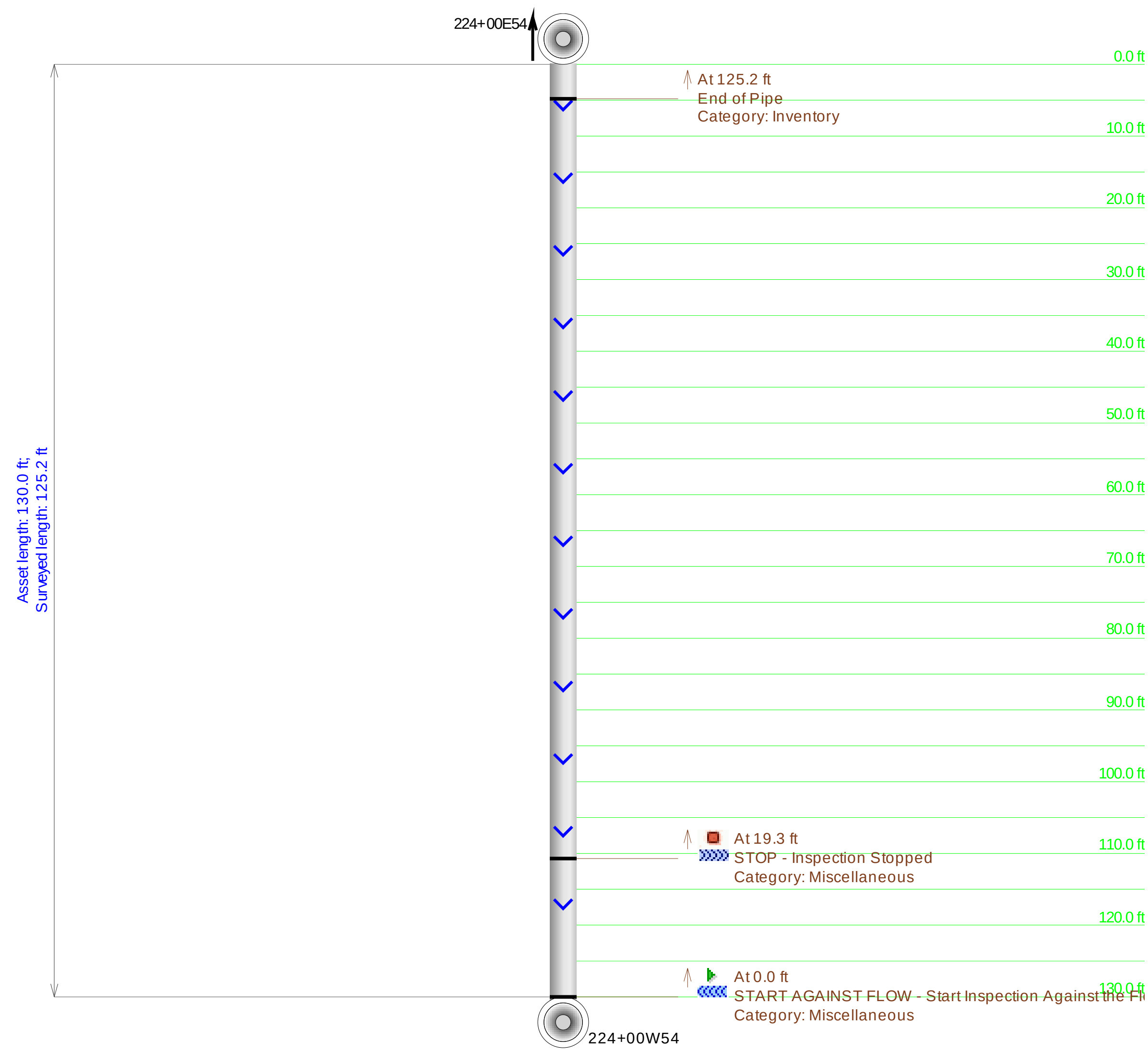


# TV Inspection with Pipe-Run Graph

| Project Name: | Mainline ID: | City: | Address: |
|---|---|---|---|
| Auburn University Research | 224+00-54-W-E20 | Auburn Al | Cox Rd |

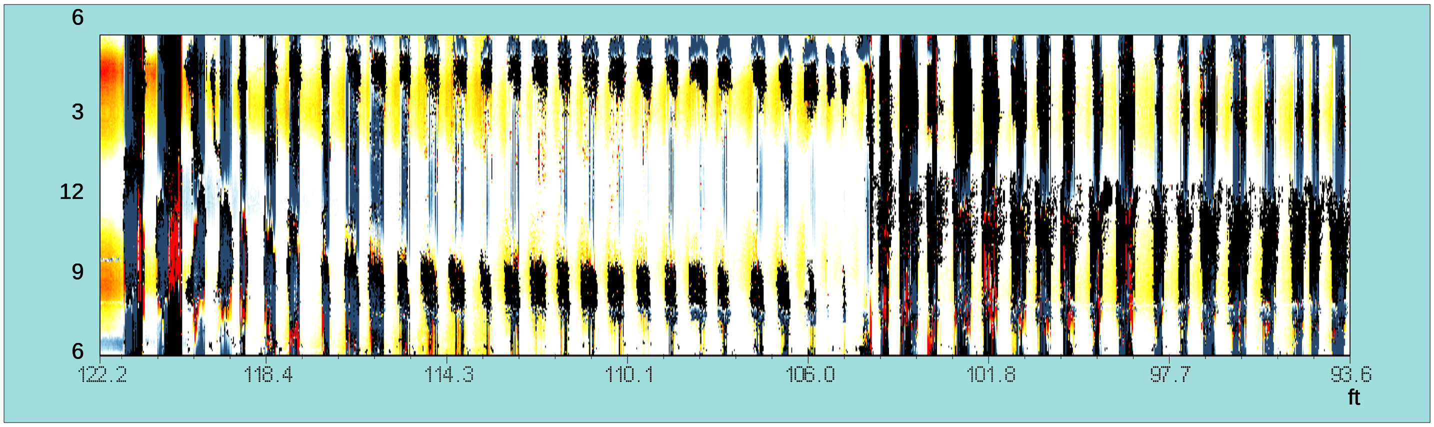
| Start date/time: | Pipe width: | Pipe height: | Pipe type: | Surface condition: |
|---|---|---|---|---|
| 7/28/2016 | 54 | 54 | Polyethylene | Asphalt |

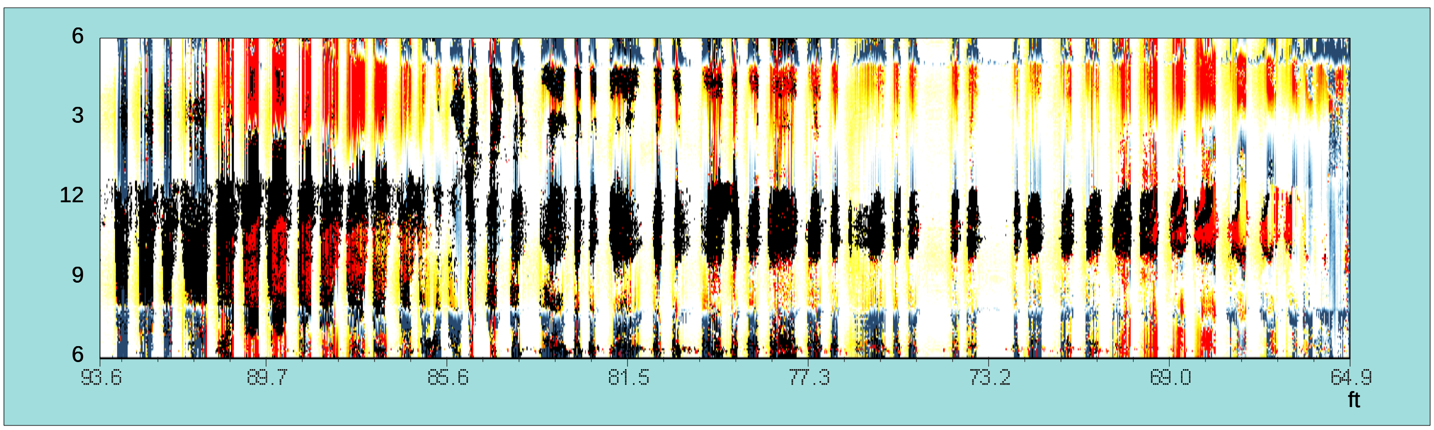
| Direction: | Surveyed footage: | Weather: | MediaLabel |
|---|---|---|---|
| Upstream | 125.2 | Dry | |

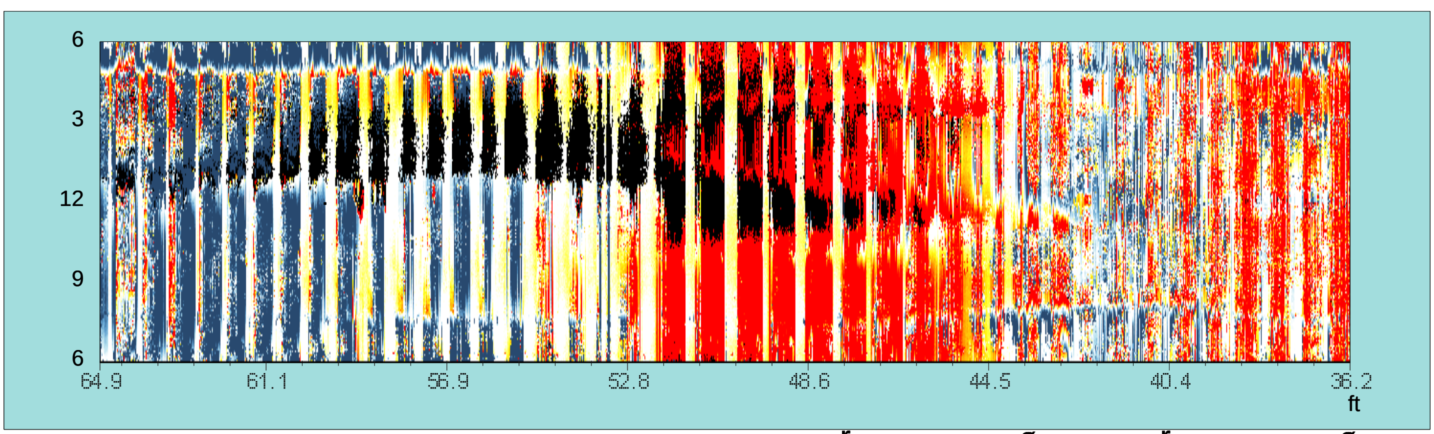


# Flat Analysis Report

| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 224+00-54 | Date | 8/8/2016 |
| City | Auburn AL | | | Material | HDPE |
| Start No | 224+00W | Finish No | 224+00E | Pipeline Length | 125.2ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 54in |



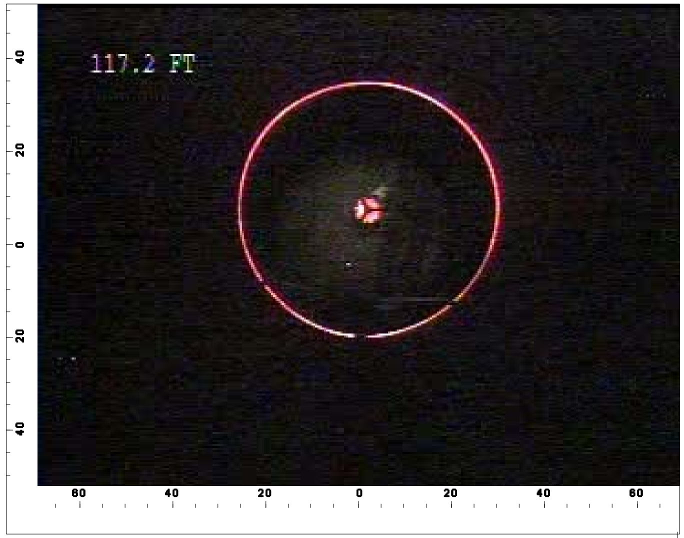

Comments

| r | | ø | r | | ø |
|---|---|---|---|---|---|
| 10.0% | 2.7in | 5.0% | -3.3% | -0.9in | -1.7% |
| 6.7% | 1.8in | 3.3% | -6.7% | -1.8in | -3.3% |
| 3.3% | 0.9in | 1.7% | -10.0% | -2.7in | -5.0% |
| 0% | 0.0in | 0% | | | |

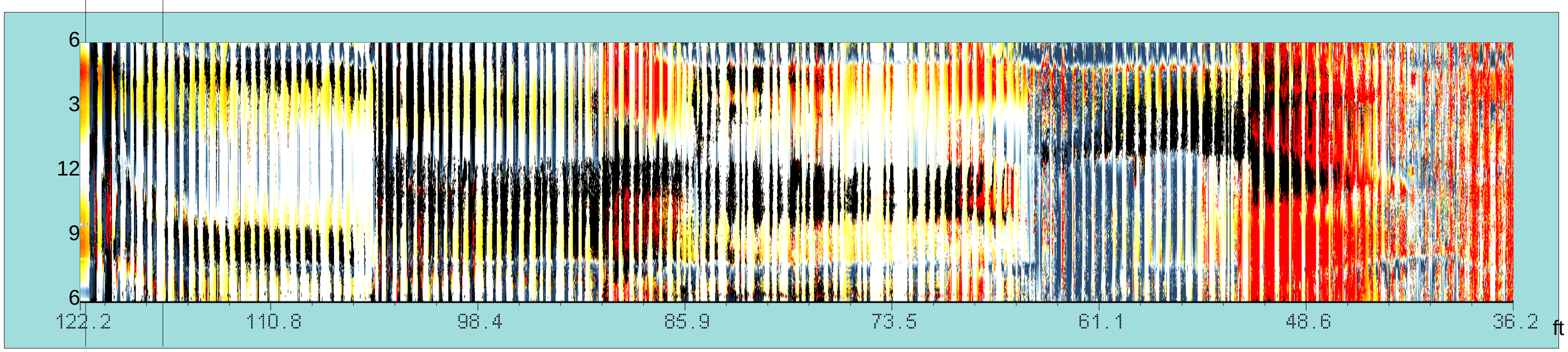
# Flat Observations Report

| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 224+00-54 | Date | 8/8/2016 |
| City | Auburn AL | | | Material | HDPE |
| Start No | 224+00W | Finish No | 224+00E | Pipeline Length | 125.2 ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 54 in |


6.8% Ovality deformation



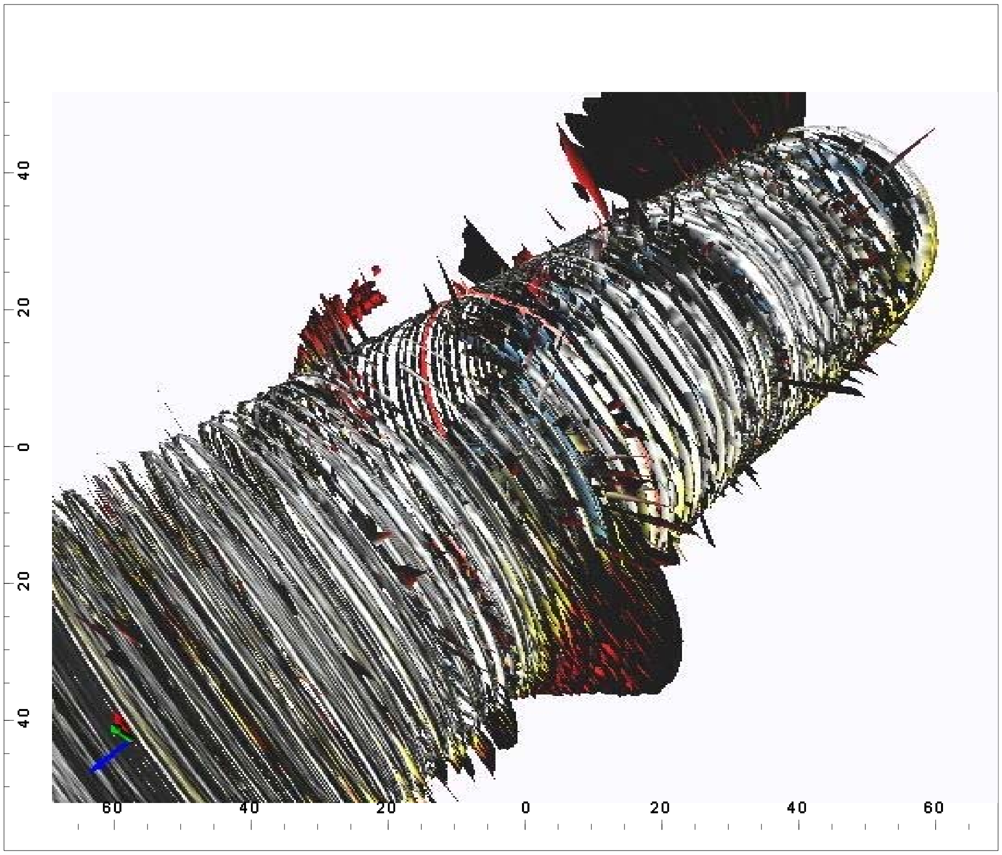

1.4% Ovality Deformation




| r | | ø |
|---|---|---|
| 10.0% | 2.7in | 5.0% |
| 6.7% | 1.8in | 3.3% |
| 3.3% | 0.9in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.9in | -1.7% |
| -6.7% | -1.8in | -3.3% |
| -10.0% | -2.7in | -5.0% |

# Flat Profile Report

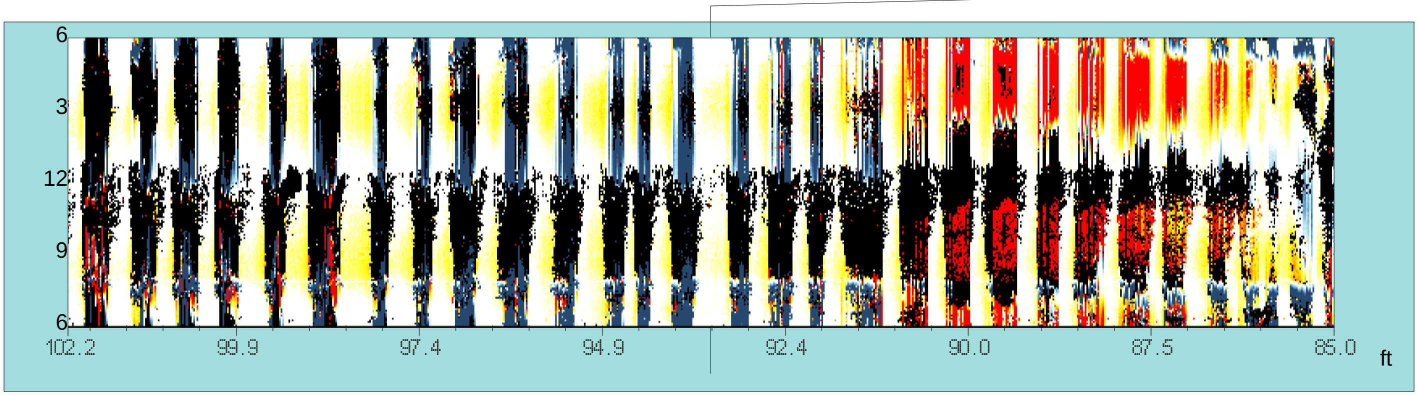
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 224+00-54 | Date | 8/8/2016 |
| City | Auburn AL | | | Material | HDPE |
| Start No | 224+00W | Finish No | 224+00E | Pipeline Length | 125.2 ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 54 in |




| r | | ø |
|---|---|---|
| 10.0% | 2.7in | 5.0% |
| 6.7% | 1.8in | 3.3% |
| 3.3% | 0.9in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.9in | -1.7% |
| -6.7% | -1.8in | -3.3% |
| -10.0% | -2.7in | -5.0% |

| | |
|---|---|
| Horizontal 55.5in 2.8% | True 55.2in |
| Vertical 54.9in 1.7% | Ovality 4.7% |
| Capacity 104.49in$^2$ 4.6% | |
| Max 57.9in 7.2% | |
| Min 54.5in 0.9% | |

ft 93ft



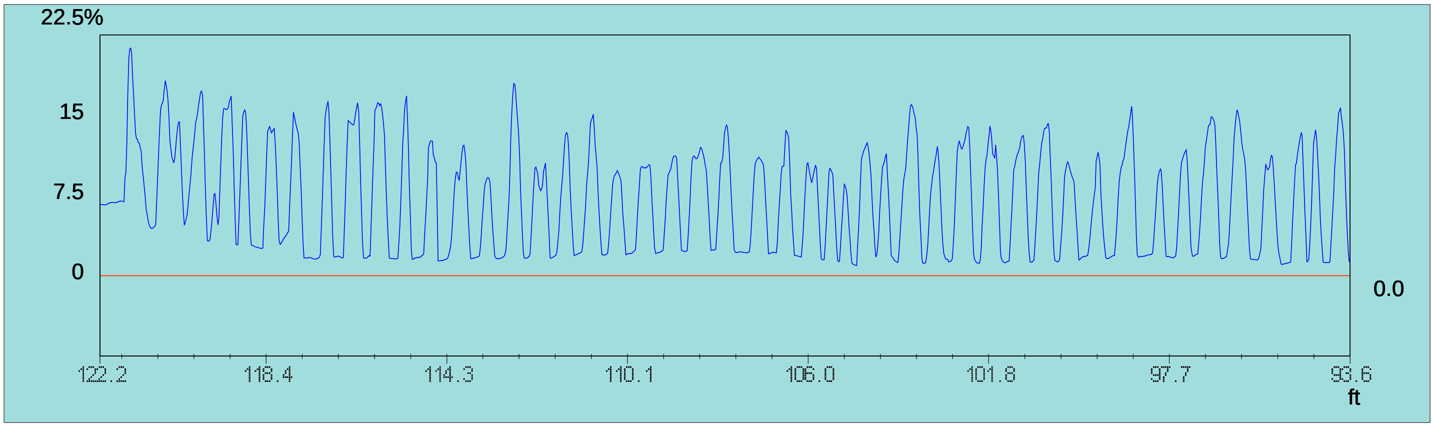

Comments

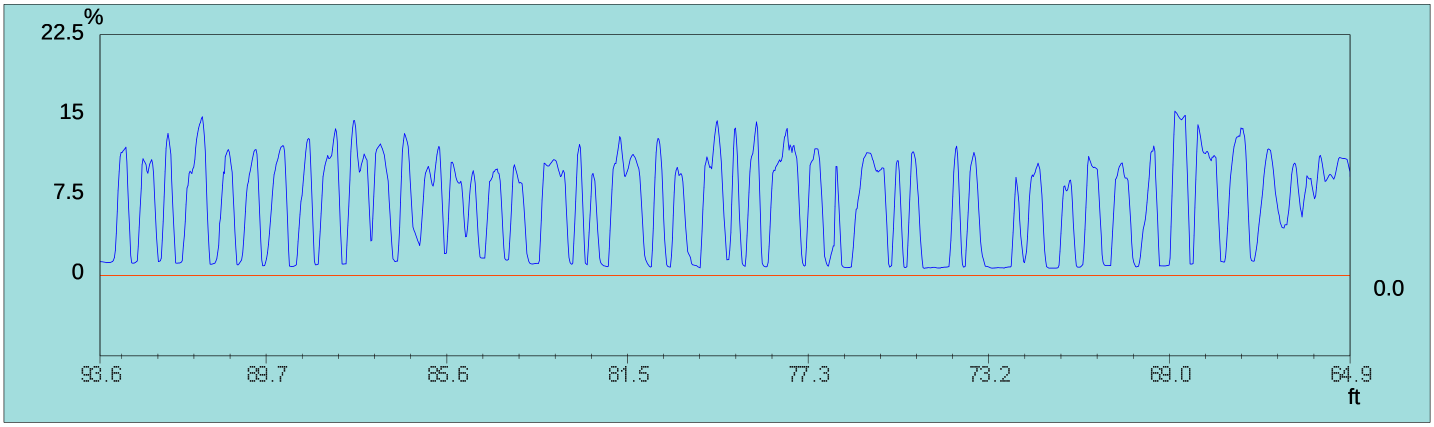
# Ovality Analysis Report

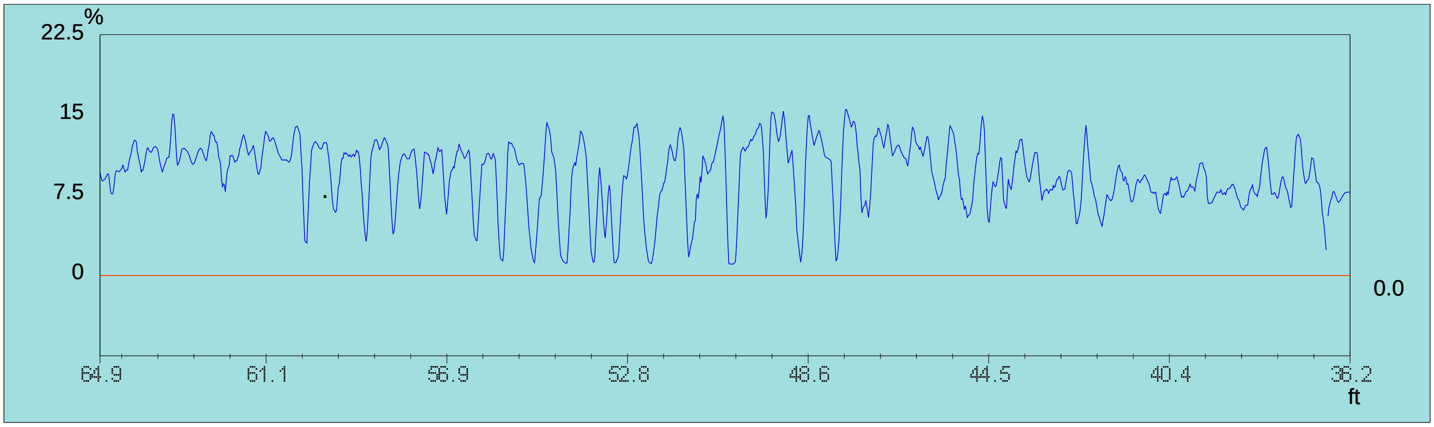
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 224+00-54 | Date | 8/8/2016 |
| City | Auburn AL | | | Material | HDPE |
| Start No | 224+00W | Finish No | 224+00E | Pipeline Length | 125.2ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 54in |

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

90% - Fractile: 12.8%, Exceeded limits: 100.0%



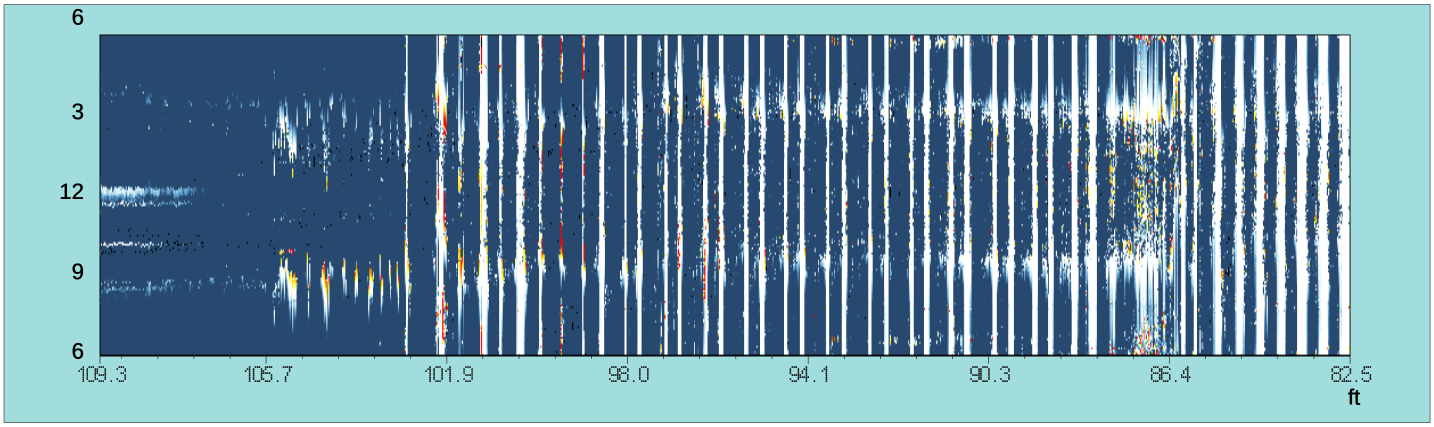



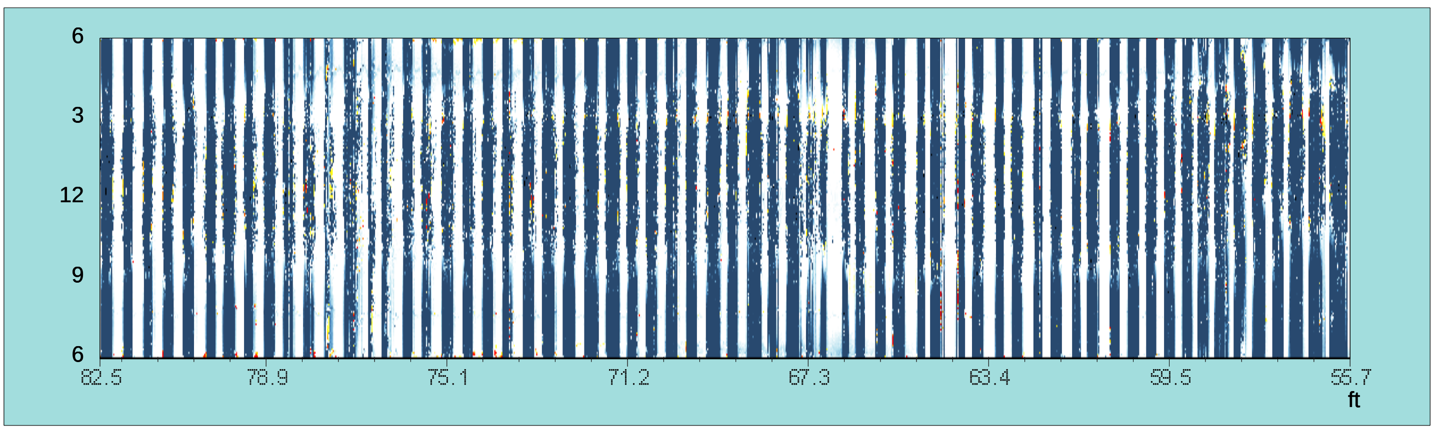

Comments

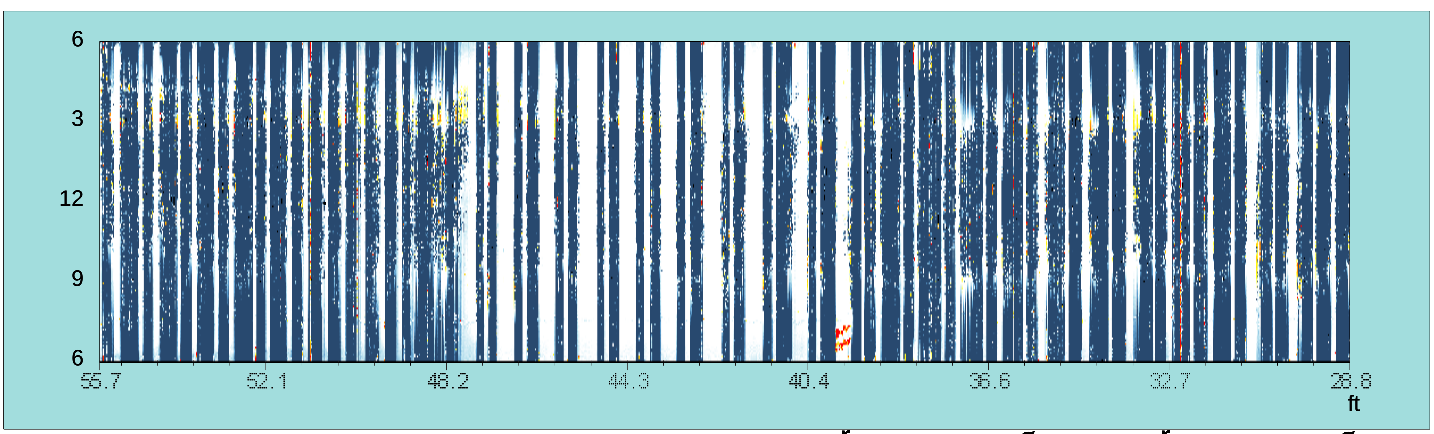
r
10.(
6.7
3.3
0%

# Flat Analysis Report

| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 224+00-48 | Date | 8/8/2016 |
| City | Auburn AL | | | Material | HDPE |
| Start No | 224+00W | Finish No | 224+00E | Pipeline Length | 123.2ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 48in |



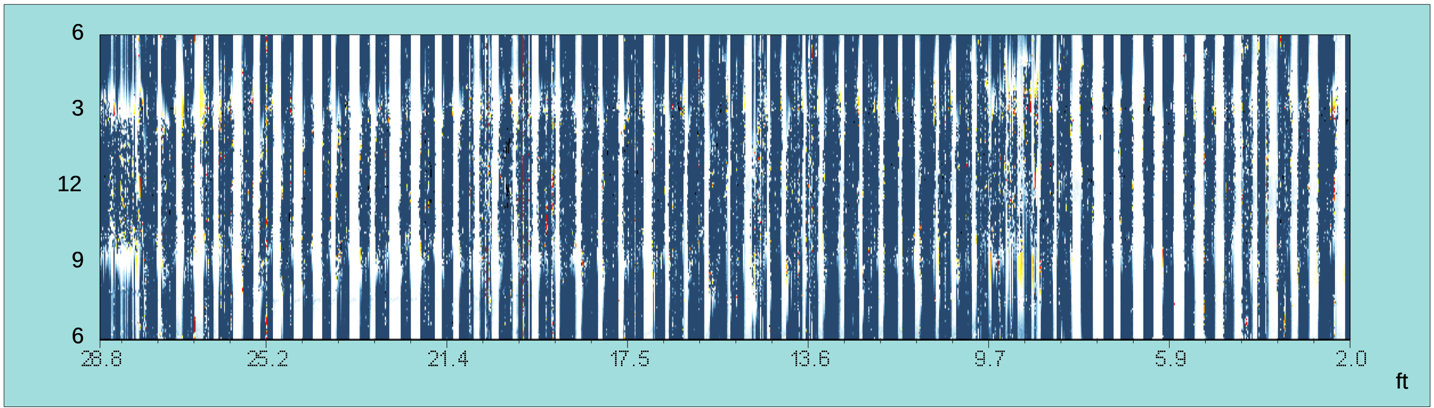




Comments

| r | | ø | r | | ø |
|---|---|---|---|---|---|
| 10.0% | 2.4in | 5.0% | -3.3% | -0.8in | -1.7% |
| 6.7% | 1.6in | 3.3% | -6.7% | -1.6in | -3.3% |
| 3.3% | 0.8in | 1.7% | -10.0% | -2.4in | -5.0% |
| 0% | 0.0in | 0% | | | |

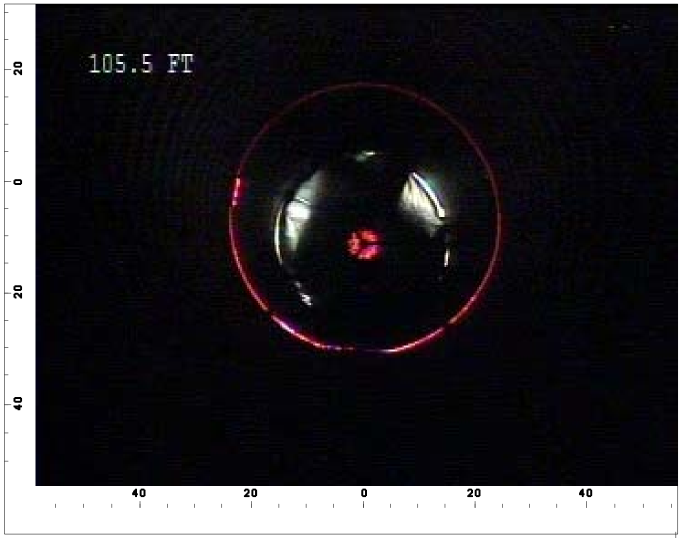
# Flat Analysis Report




| r | ø | r | ø |
|---|---|---|---|
| 10.0% | | -3.3% | |
| 6.7% | | -6.7% | |
| 3.3% | | -10.0% | |
| 0% | | | |

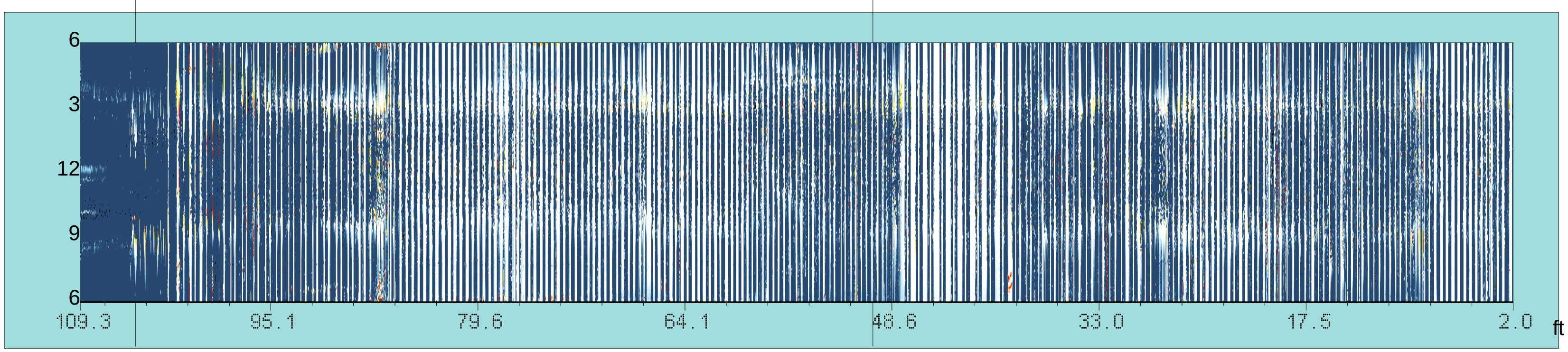
# Flat Observations Report

| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 224+00-48 | Date | 8/8/2016 |
| City | Auburn AL | | | Material | HDPE |
| Start No | 224+00W | Finish No | 224+00E | Pipeline Length | 123.2 ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 48 in |




1.2% Ovality Deformation

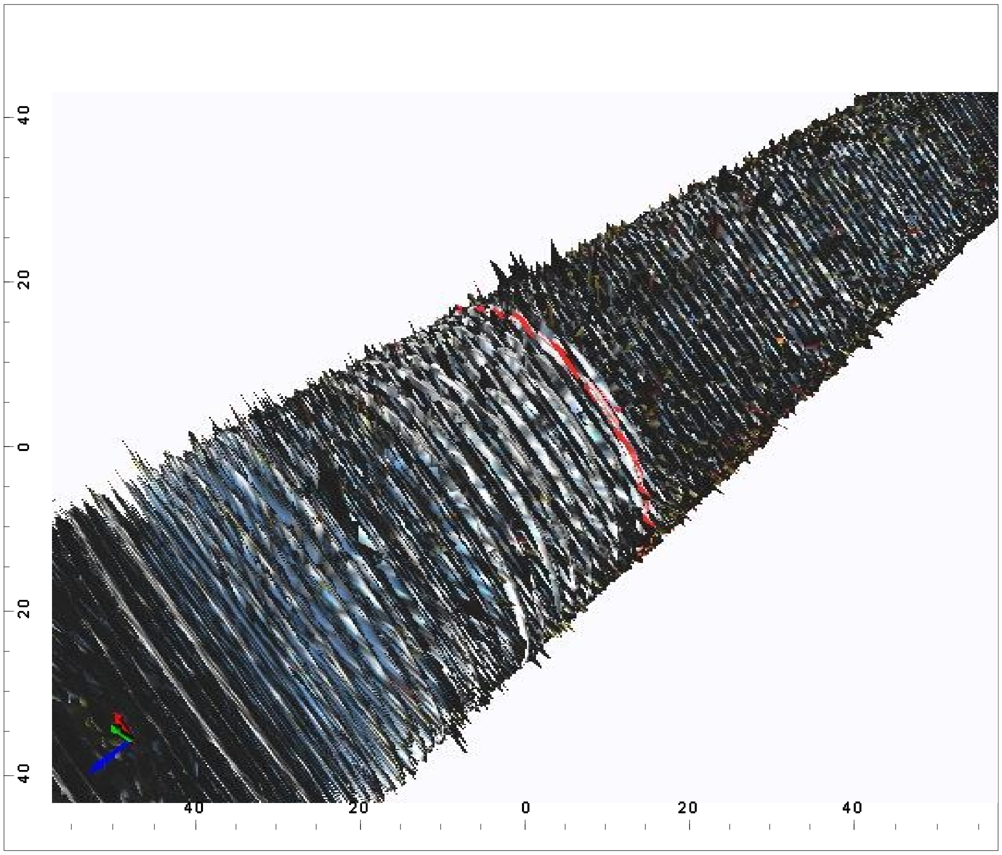

0.0% Ovality Deformation




| r | | ø |
|---|---|---|
| 10.0% | 2.4in | 5.0% |
| 6.7% | 1.6in | 3.3% |
| 3.3% | 0.8in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.8in | -1.7% |
| -6.7% | -1.6in | -3.3% |
| -10.0% | -2.4in | -5.0% |

# Flat Profile Report

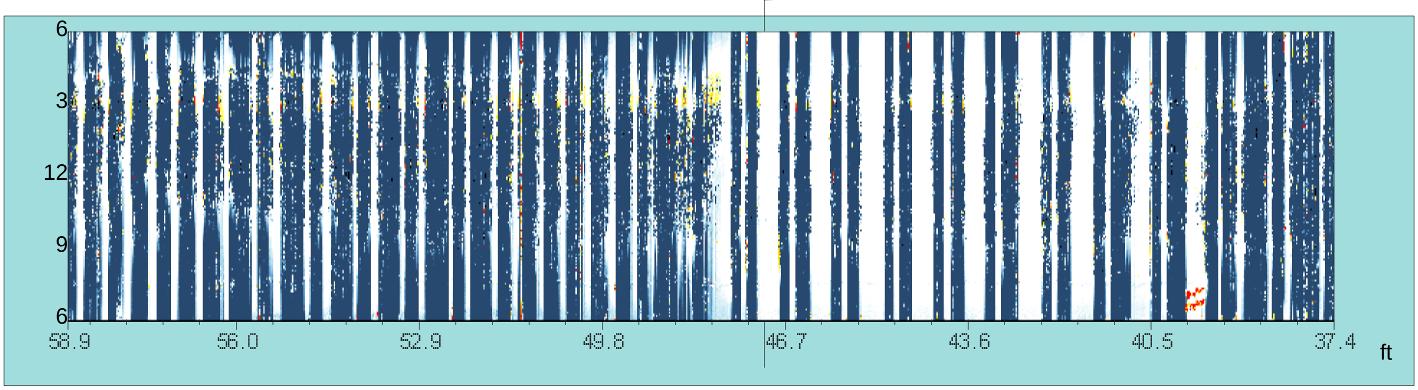
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 224+00-48 | Date | 8/8/2016 |
| City | Auburn AL | | | Material | HDPE |
| Start No | 224+00W | Finish No | 224+00E | Pipeline Length | 123.2 ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 48 in |




| r | | ø |
|---|---|---|
| 10.0% | 2.4in | 5.0% |
| 6.7% | 1.6in | 3.3% |
| 3.3% | 0.8in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.8in | -1.7% |
| -6.7% | -1.6in | -3.3% |
| -10.0% | -2.4in | -5.0% |

| | |
|---|---|
| Horizontal 48.7in 1.5% | True 47.9in |
| Vertical 47.1in -1.9% | Ovality 2.0% |
| Capacity $-4.28in^2$ -0.2% | |
| Max 48.7in 1.5% | |
| Min 47.0in -2.0% | |

ft 47ft



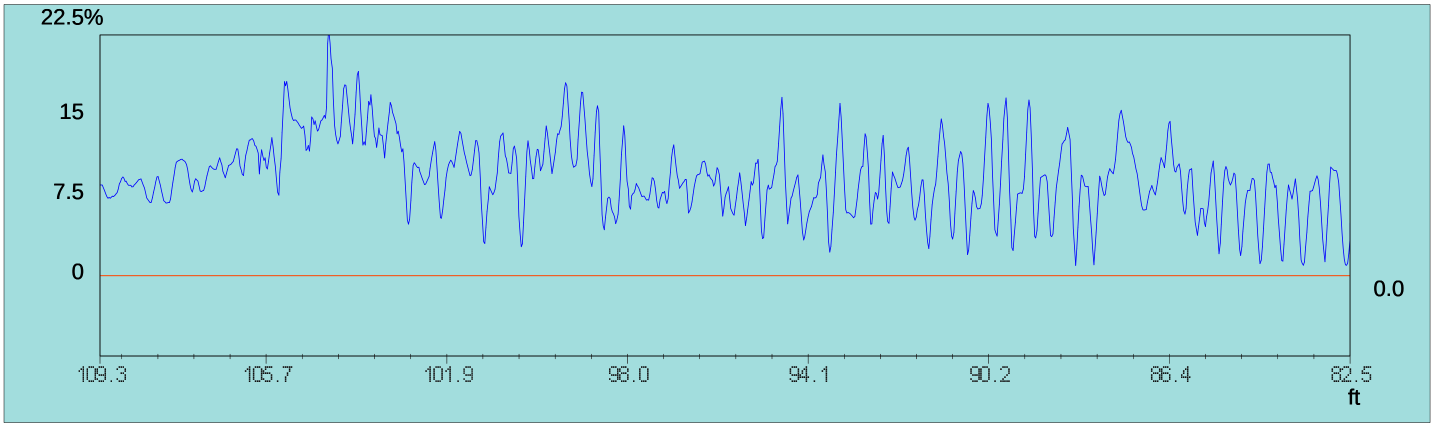

Comments

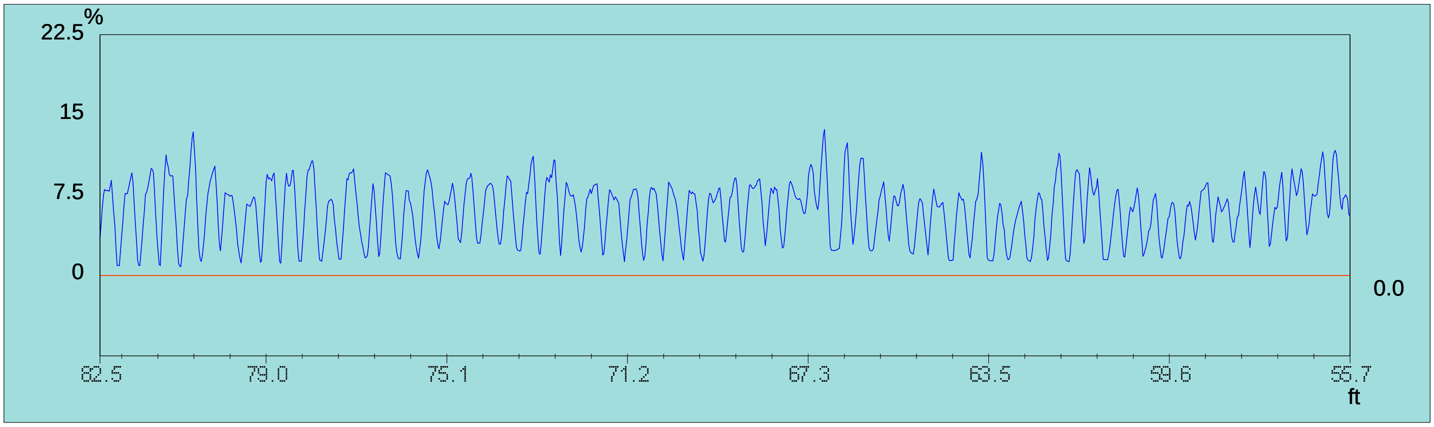
# Ovality Analysis Report

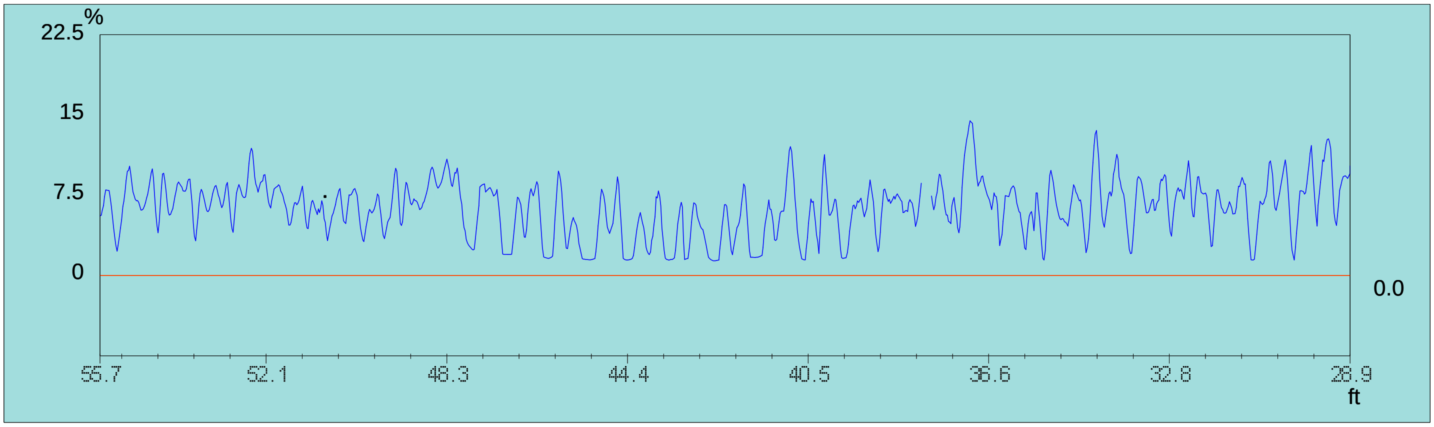
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 224+00-48 | Date | 8/8/2016 |
| City | Auburn AL | | | Material | HDPE |
| Start No | 224+00W | Finish No | 224+00E | Pipeline Length | 123.2ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 48in |

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

90% - Fractile: 10.8%, Exceeded limits: 100.0%



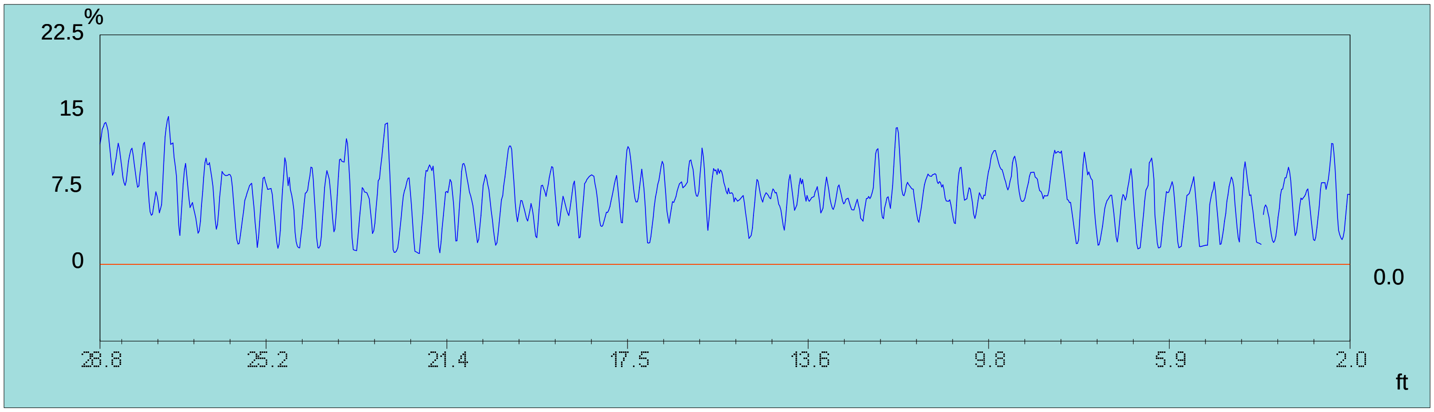




Comments

r
10.
6.7
3.3
0%

# Ovality Analysis Report

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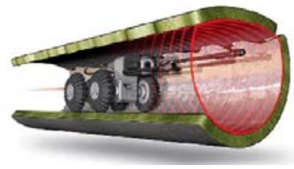
Liberty, SC 29657
1-800-990-8406

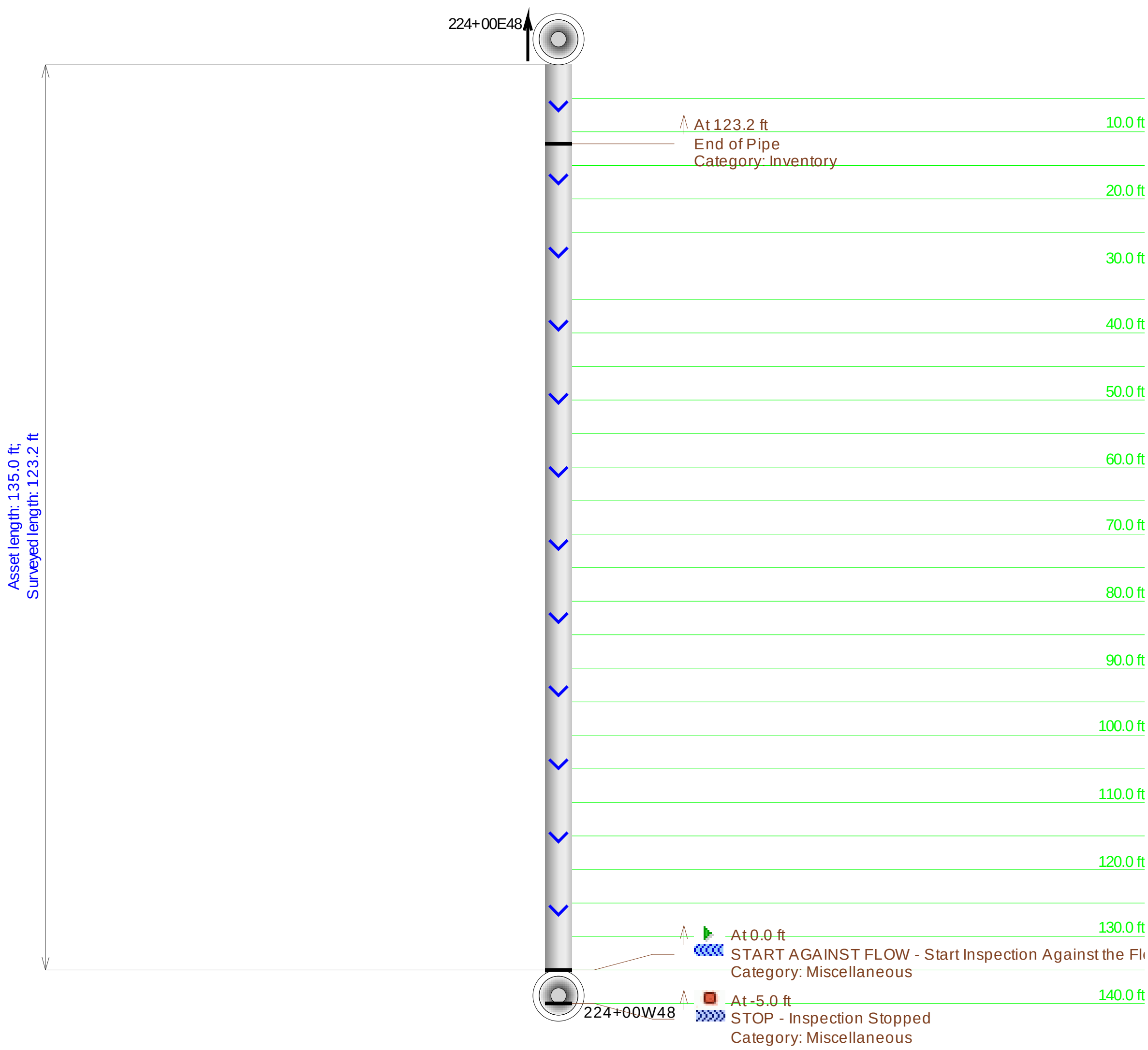


# TV Inspection with Pipe-Run Graph

| Project Name: | Mainline ID: | City: | Address: |
|---|---|---|---|
| Auburn University Research | 224+00-48-W-E | Auburn Al | Cox Rd |

| Start date/time: | Pipe width: | Pipe height: | Pipe type: | Surface condition: |
|---|---|---|---|---|
| 7/28/2016 | 48 | 48 | Polyethylene | Asphalt |

| Direction: | Surveyed footage: | Weather: | MediaLabel |
|---|---|---|---|
| Upstream | 123.2 | Dry | |



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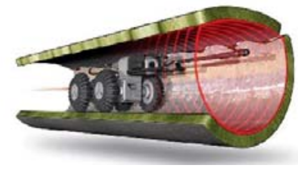
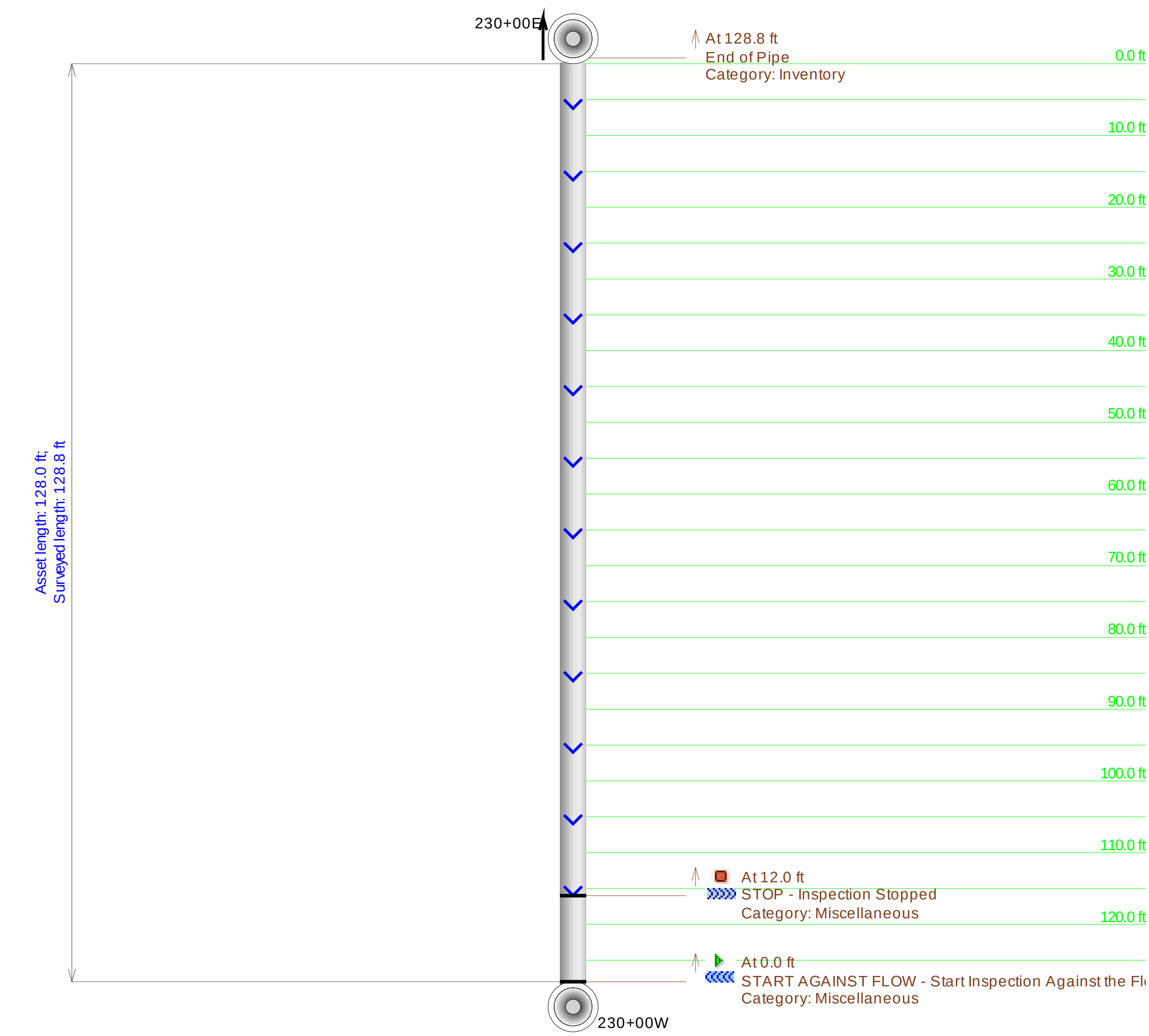


# TV Inspection with Pipe-Run Graph

| Project Name: | Mainline ID: | City: | Address: |
|---|---|---|---|
| Auburn University Research | 230+00W-E | Auburn Al | Cox Rd |

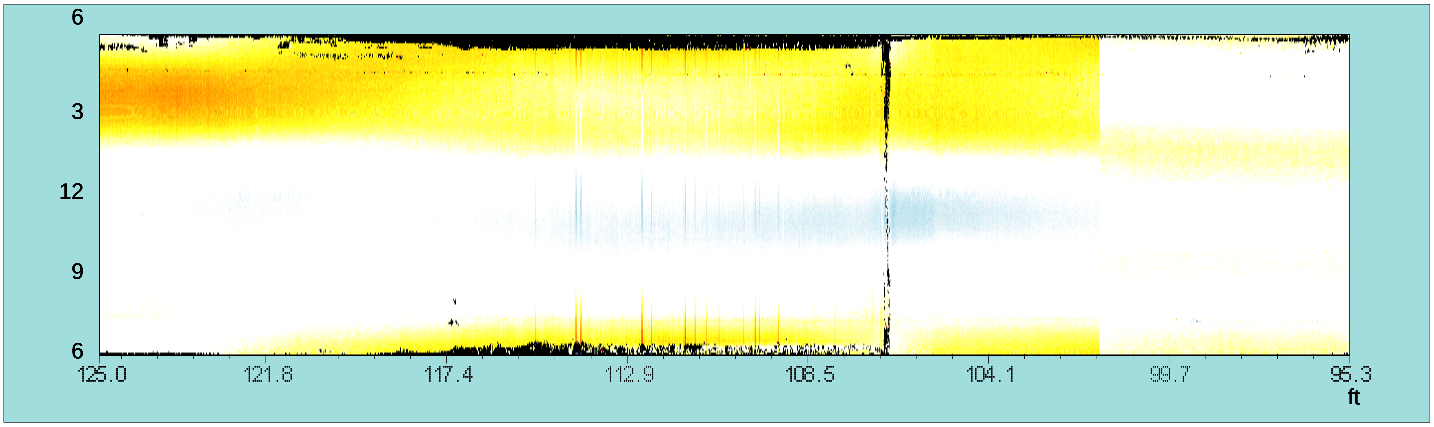
| Start date/time: | Pipe width: | Pipe height: | Pipe type: | Surface condition: |
|---|---|---|---|---|
| 7/28/2016 | 36 | 36 | PVC | Asphalt |

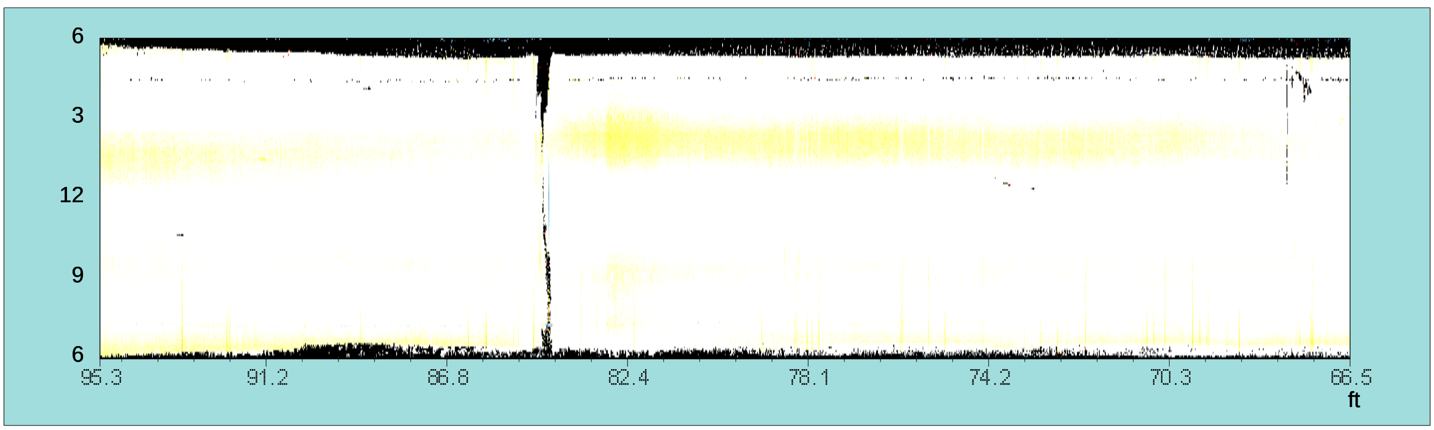
| Direction: | Surveyed footage: | Weather: | MediaLabel |
|---|---|---|---|
| Upstream | 128.8 | Dry | |

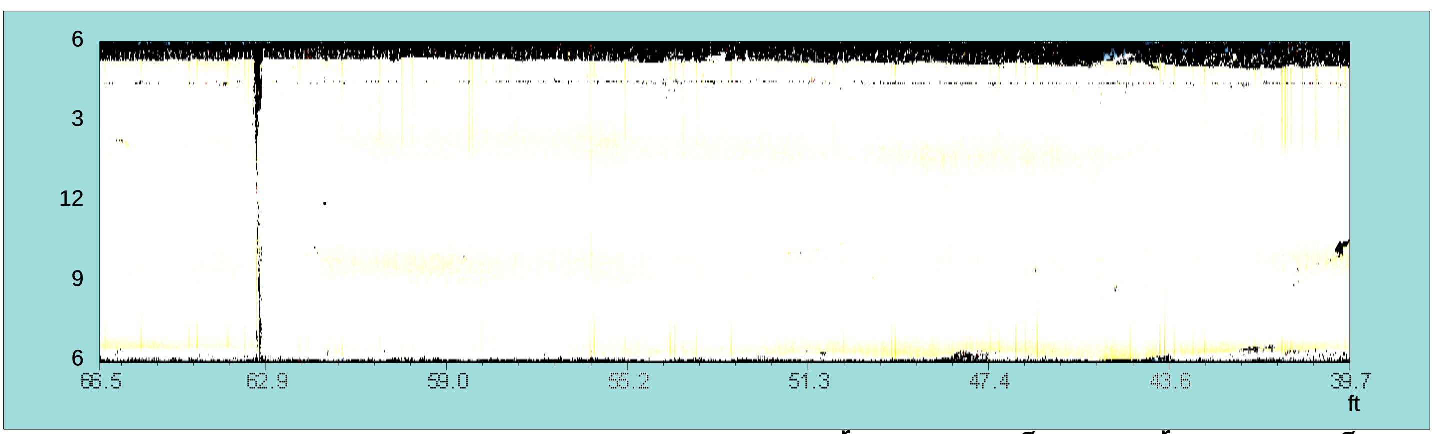


# Flat Analysis Report

| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 230+00 | Date | 8/8/2016 |
| City | Auburn Al | | | Material | P VC |
| Start No | 230+00W | Finish No | 230+00E | Pipeline Length | 128ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87in |



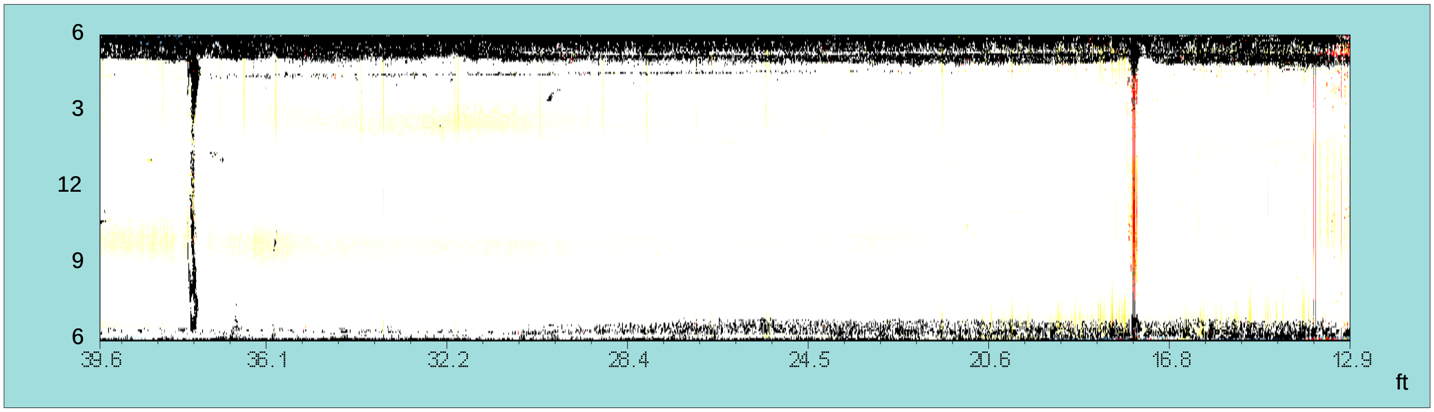




Comments

| r | | ø | r | | ø |
|---|---|---|---|---|---|
| 10.0% | 1.8in | 5.0% | -3.3% | -0.6in | -1.7% |
| 6.7% | 1.2in | 3.3% | -6.7% | -1.2in | -3.3% |
| 3.3% | 0.6in | 1.7% | -10.0% | -1.8in | -5.0% |
| 0% | 0.0in | 0% | | | |

# Flat Analysis Report

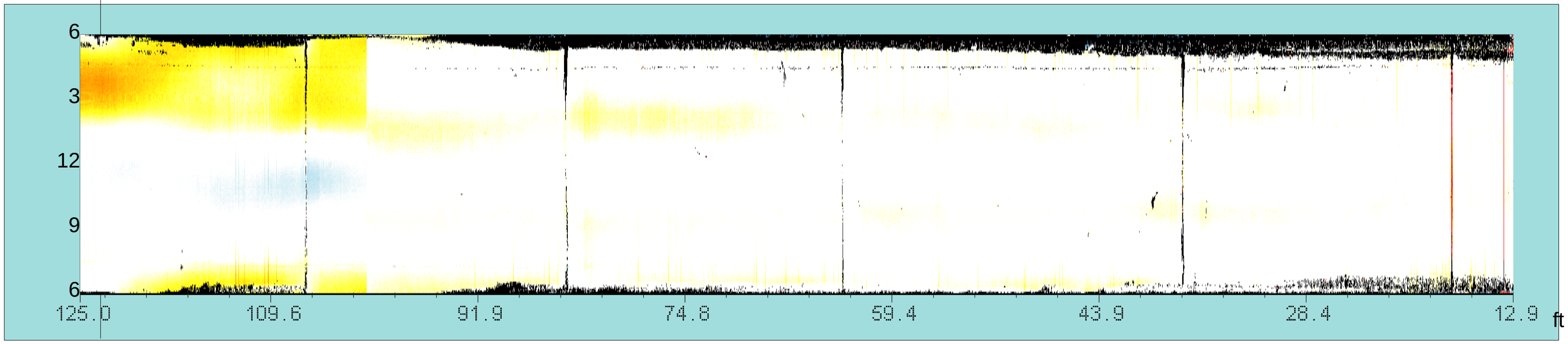



| r | ø | r | ø |
|---|---|---|---|
| 10.0% | | -3.3% | |
| 6.7% | | -6.7% | |
| 3.3% | | -10.0% | |
| 0% | | | |

# Flat Observations Report

| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 230+00 | Date | 8/8/2016 |
| City | Auburn Al | | | Material | P VC |
| Start No | 230+00W | Finish No | 230+00E | Pipeline Length | 128 ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87 in |

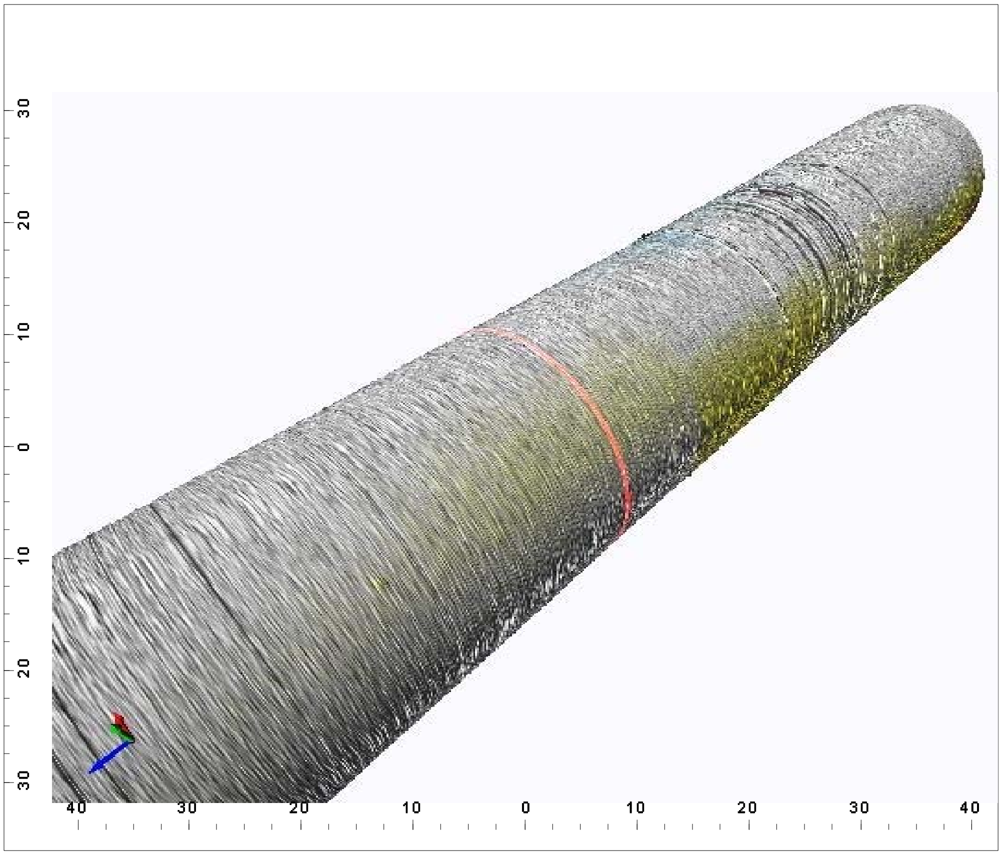

2.1% Ovality deformation




| r | | ø |
|---|---|---|
| 10.0% | 1.8in | 5.0% |
| 6.7% | 1.2in | 3.3% |
| 3.3% | 0.6in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.6in | -1.7% |
| -6.7% | -1.2in | -3.3% |
| -10.0% | -1.8in | -5.0% |

# Flat Profile Report

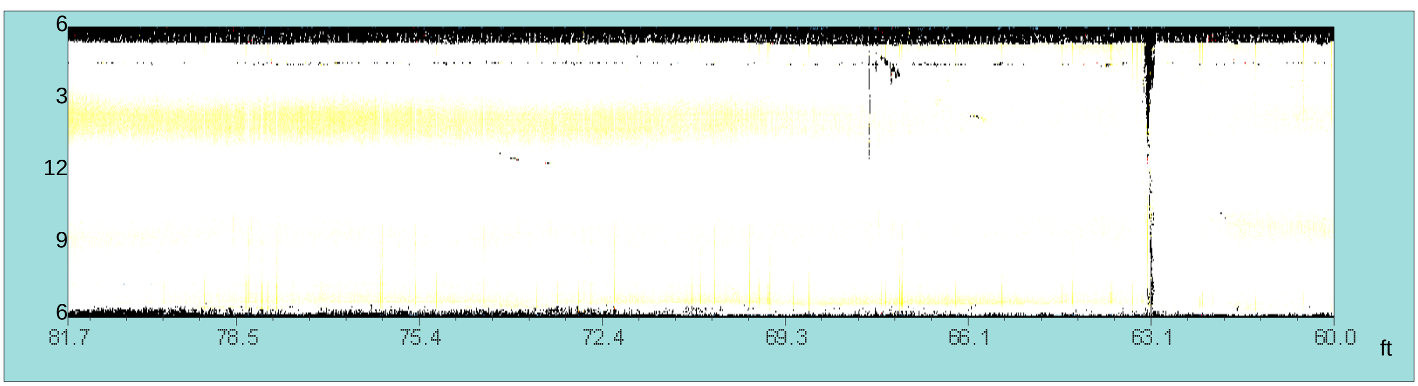
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 230+00 | Date | 8/8/2016 |
| City | Auburn Al | | | Material | P VC |
| Start No | 230+00W | Finish No | 230+00E | Pipeline Length | 128 ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87 in |




| r | | ø |
|---|---|---|
| 10.0% | 1.8in | 5.0% |
| 6.7% | 1.2in | 3.3% |
| 3.3% | 0.6in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.6in | -1.7% |
| -6.7% | -1.2in | -3.3% |
| -10.0% | -1.8in | -5.0% |

| | | | |
|---|---|---|---|
| Horizontal | 36.5in 1.7% | True | 36.6in |
| Vertical | 36.8in 2.5% | Ovality | 1.0% |
| Capacity | 40.70in² 4.0% | | |
| Max | 36.9in 3.0% | | |
| Min | 36.2in 0.8% | | |

**ft** 98ft




Comments

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www.cleanflowsystems.com

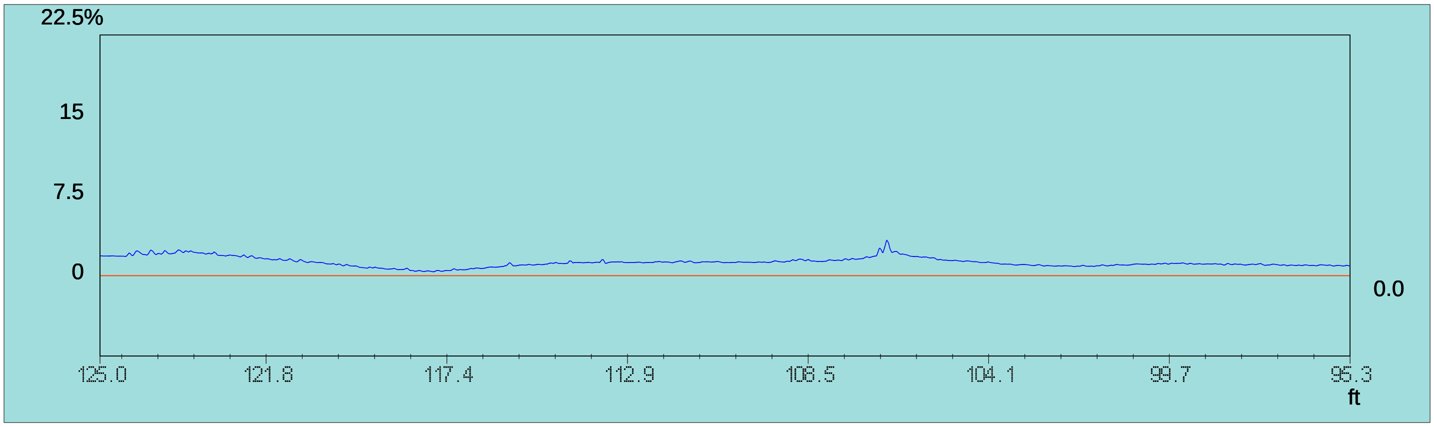
Printed: 8/9/2016

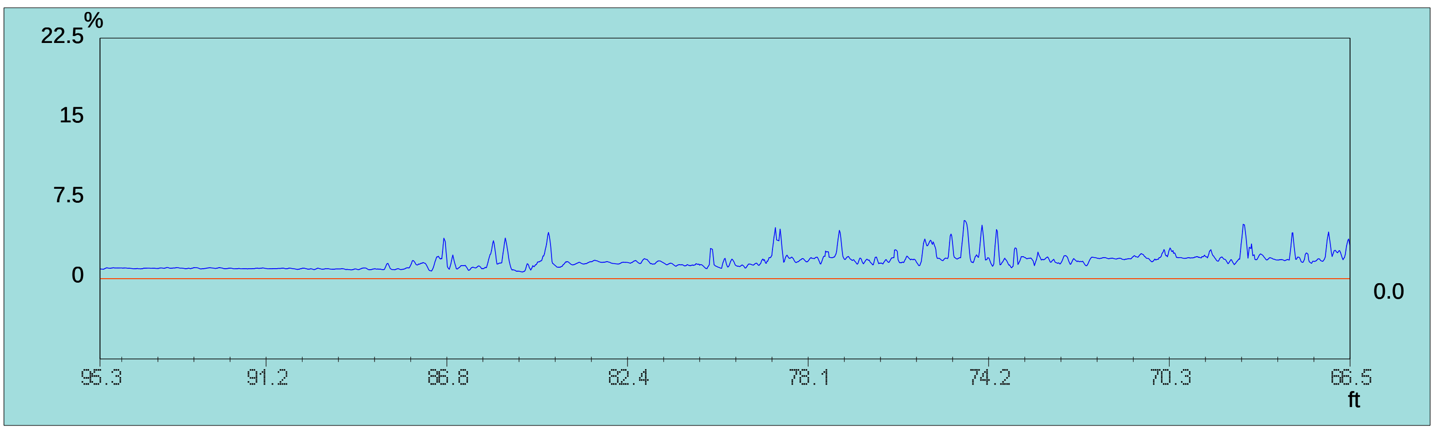
# Ovality Analysis Report

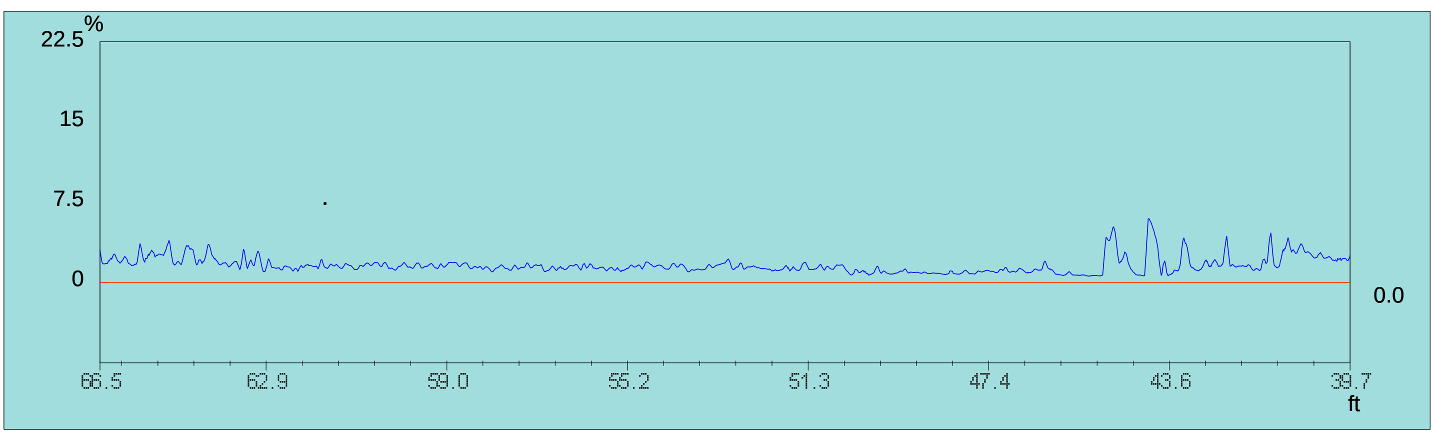
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 230+00 | Date | 8/8/2016 |
| City | Auburn Al | | | Material | P VC |
| Start No | 230+00W | Finish No | 230+00E | Pipeline Length | 128ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 35.87in |

90% - Fractile: 2.4%, Exceeded limits: 100.0%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



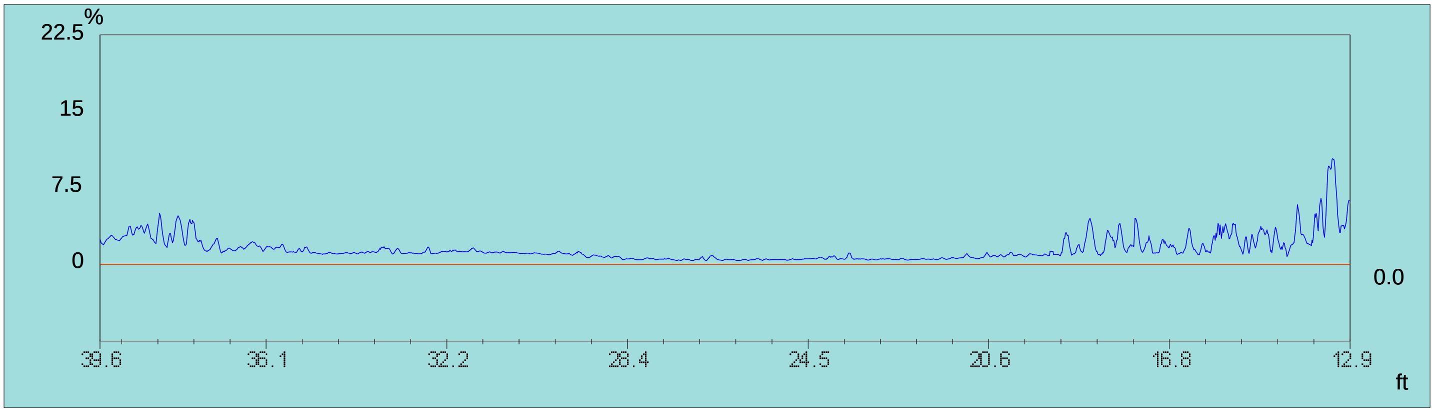




Comments

r
10.
6.7
3.3
0%

# Ovality Analysis Report



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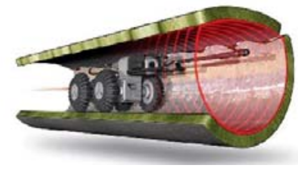
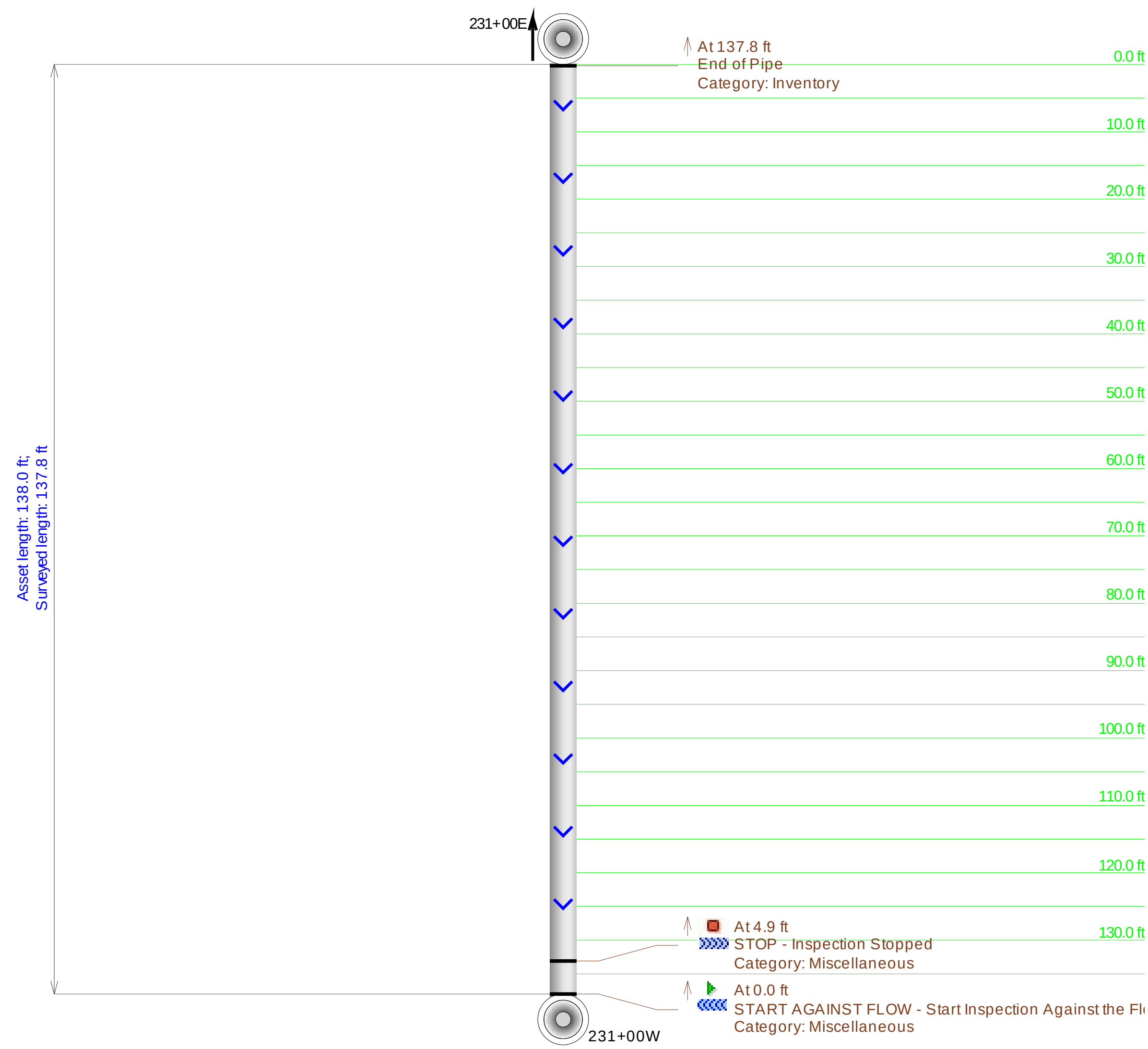


# TV Inspection with Pipe-Run Graph

| Project Name: | Mainline ID: | City: | Address: |
|---|---|---|---|
| Auburn University Research | 231+00W-E | Auburn Al | Cox Rd |

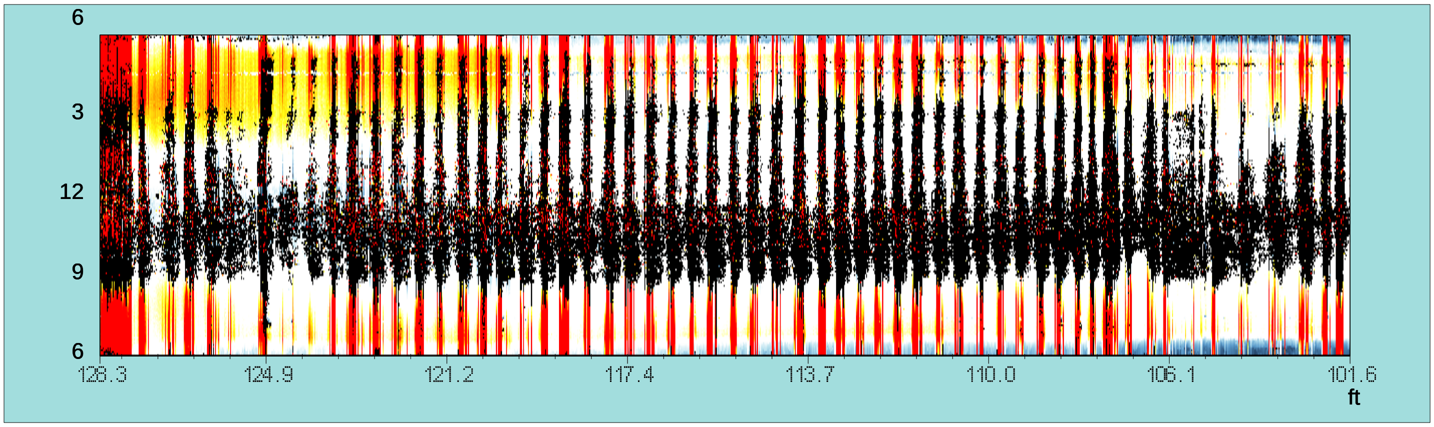
| Start date/time: | Pipe width: | Pipe height: | Pipe type: | Surface condition: |
|---|---|---|---|---|
| 7/28/2016 | 36 | 36 | Polyethylene | Asphalt |

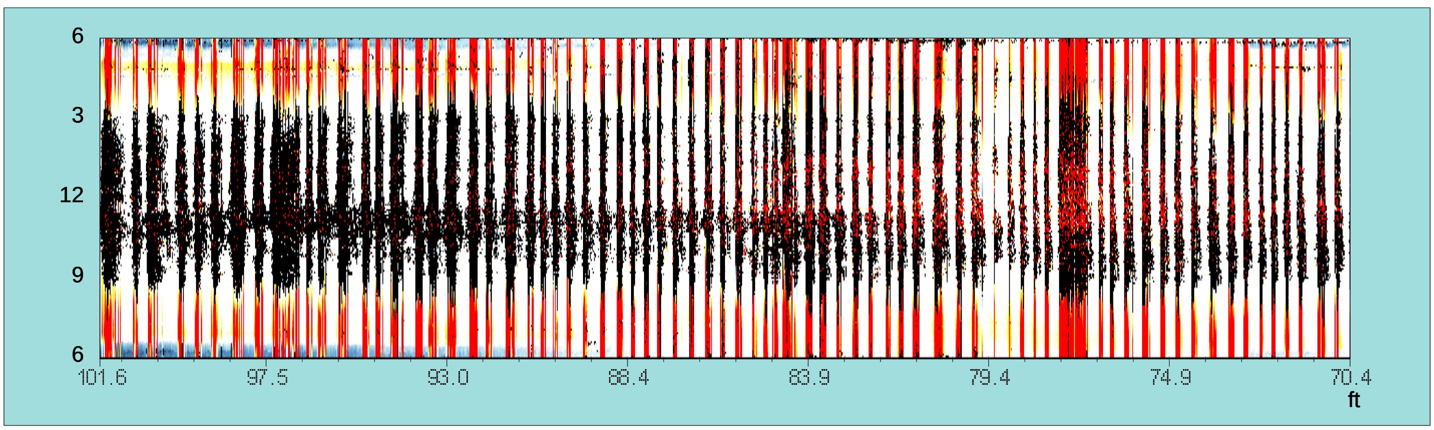
| Direction: | Surveyed footage: | Weather: | MediaLabel |
|---|---|---|---|
| Upstream | 137.8 | Dry | |

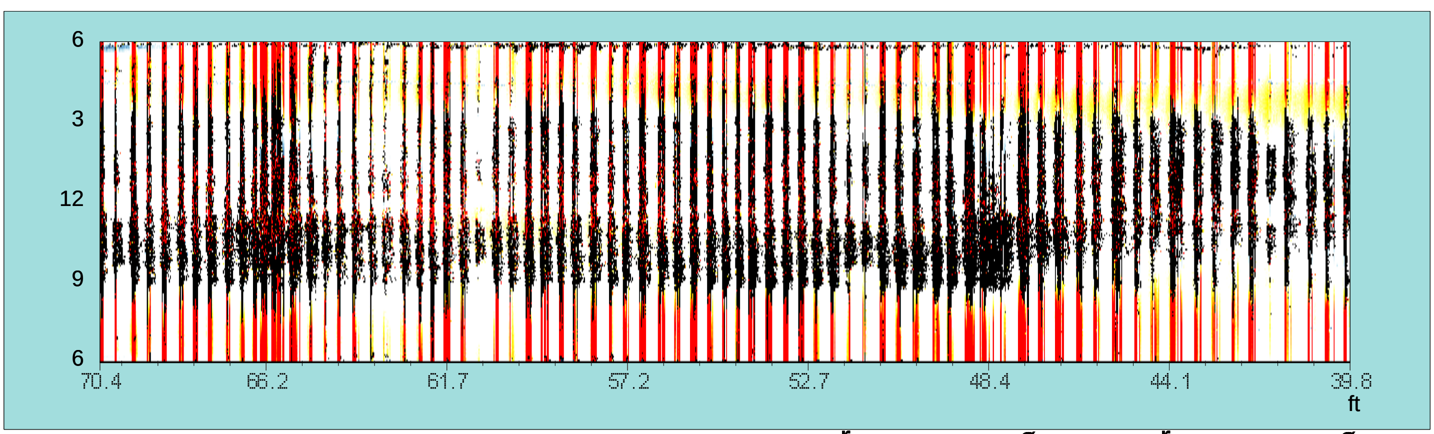


# Flat Analysis Report

| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 231+00 | Date | 8/9/2016 |
| City | Auburn AL | | | Material | HDPE |
| Start No | 231+00W | Finish No | 231+00E | Pipeline Length | 128ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 36in |



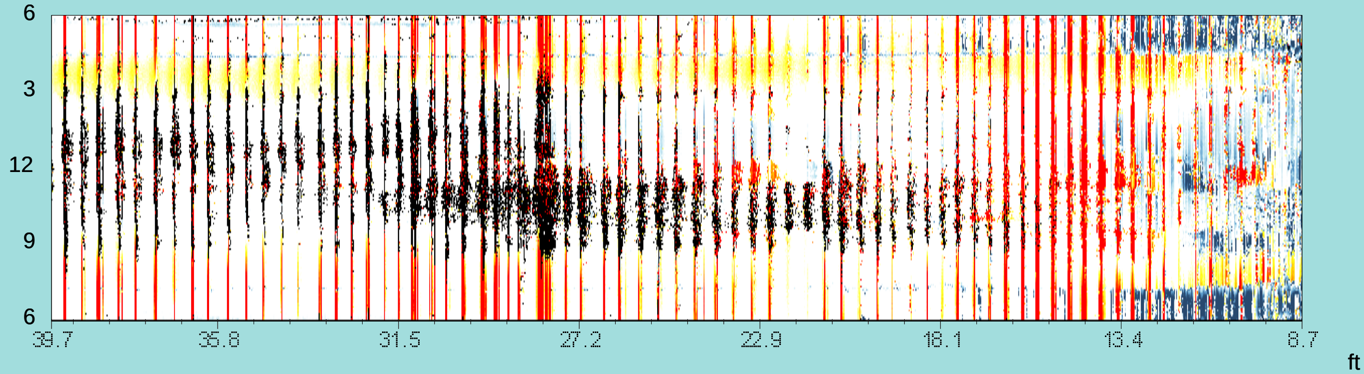




Comments

| r | | ø | r | | ø |
|---|---|---|---|---|---|
| 10.0% | 1.8in | 5.0% | -3.3% | -0.6in | -1.7% |
| 6.7% | 1.2in | 3.3% | -6.7% | -1.2in | -3.3% |
| 3.3% | 0.6in | 1.7% | -10.0% | -1.8in | -5.0% |
| 0% | 0.0in | 0% | | | |

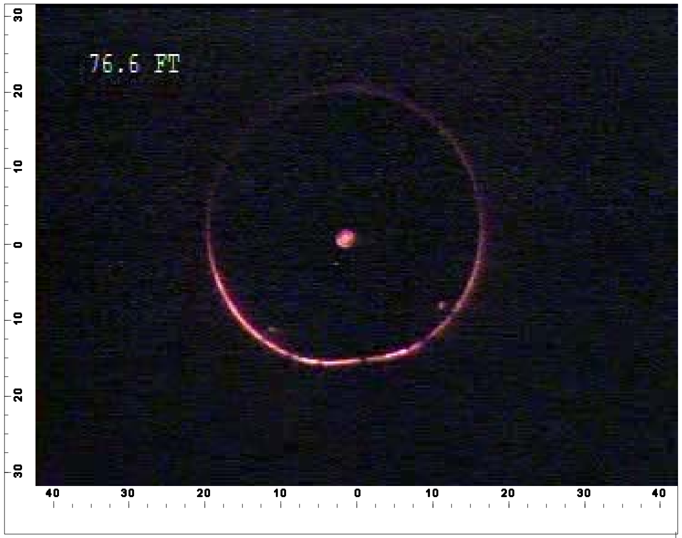
# Flat Analysis Report



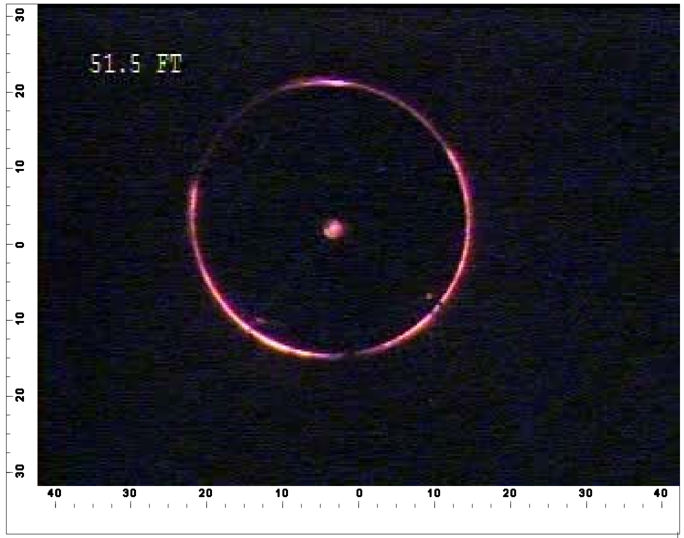

| r | ø | r | ø |
|---|---|---|---|
| 10.0% | | -3.3% | |
| 6.7% | | -6.7% | |
| 3.3% | | -10.0% | |
| 0% | | | |

# Flat Observations Report

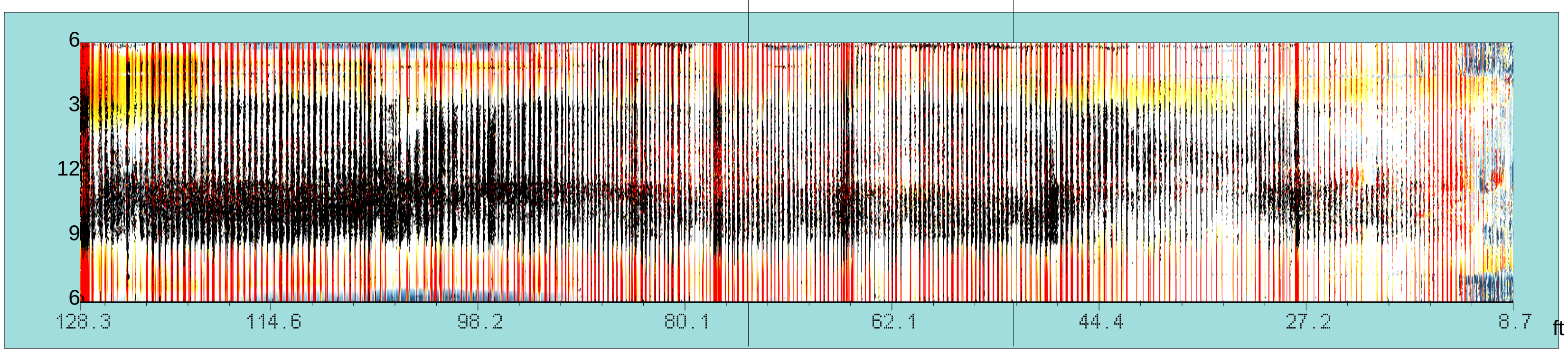
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 231+00 | Date | 8/9/2016 |
| City | Auburn AL | | | Material | HDPE |
| Start No | 231+00W | Finish No | 231+00E | Pipeline Length | 128 ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 36 in |




1.3% Ovality deformation



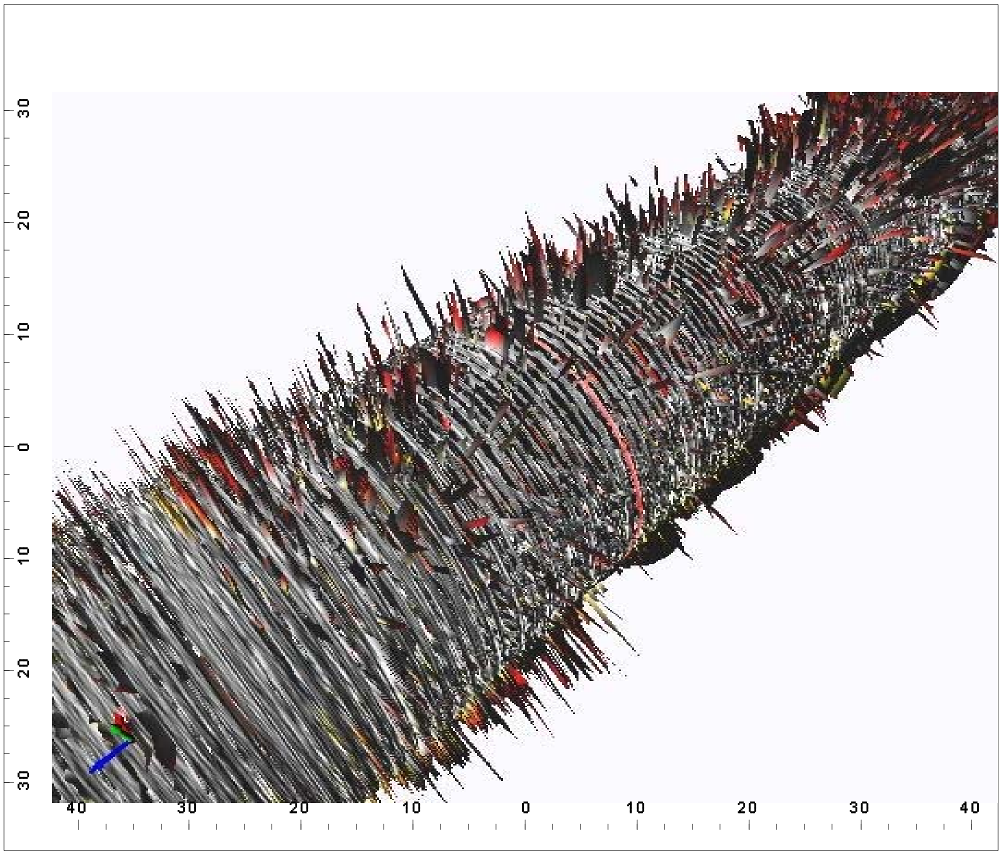

1.9%Ovality deformation




| r | | ø |
|---|---|---|
| 10.0% | 1.8in | 5.0% |
| 6.7% | 1.2in | 3.3% |
| 3.3% | 0.6in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.6in | -1.7% |
| -6.7% | -1.2in | -3.3% |
| -10.0% | -1.8in | -5.0% |

# Flat Profile Report

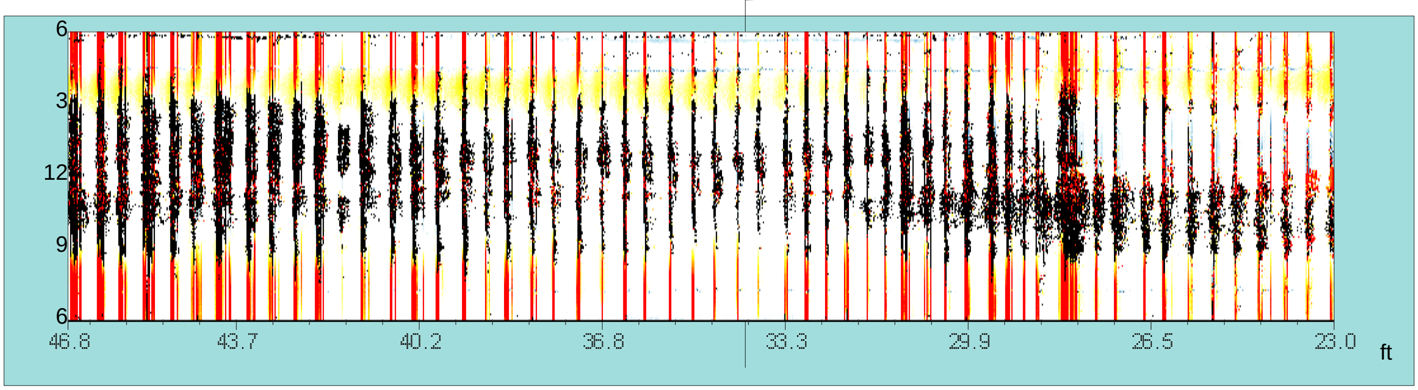
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 231+00 | Date | 8/9/2016 |
| City | Auburn AL | | | Material | HDPE |
| Start No | 231+00W | Finish No | 231+00E | Pipeline Length | 128 ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 36 in |




| r | | ø |
|---|---|---|
| 10.0% | 1.8in | 5.0% |
| 6.7% | 1.2in | 3.3% |
| 3.3% | 0.6in | 1.7% |
| 0% | 0.0in | 0% |
| -3.3% | -0.6in | -1.7% |
| -6.7% | -1.2in | -3.3% |
| -10.0% | -1.8in | -5.0% |

| | | | |
|---|---|---|---|
| Horizontal | 36.6in 1.6% | True | 36.2in |
| Vertical | 35.4in -1.6% | Ovality | 2.4% |
| Capacity | 6.47in$^2$ 0.6% | | |
| Max | 36.7in 1.8% | | |
| Min | 35.2in -2.3% | | |

ft 34ft



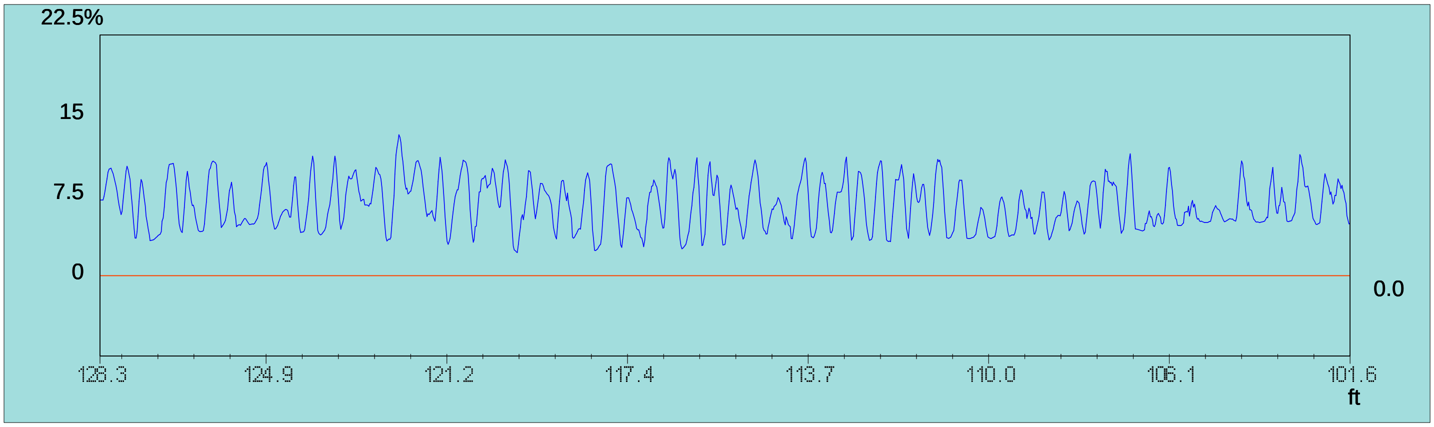

Comments

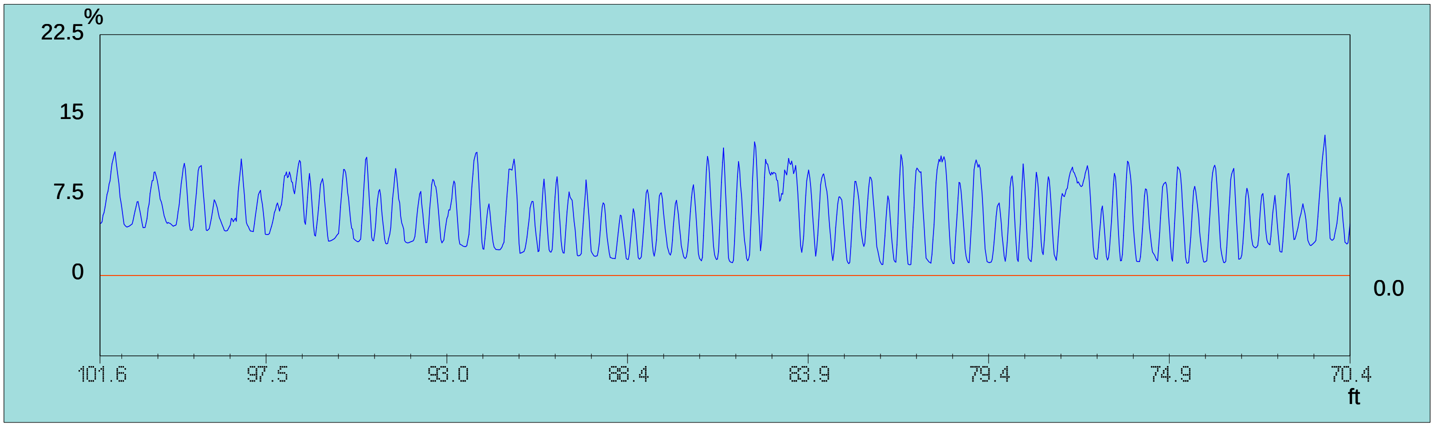
# Ovality Analysis Report

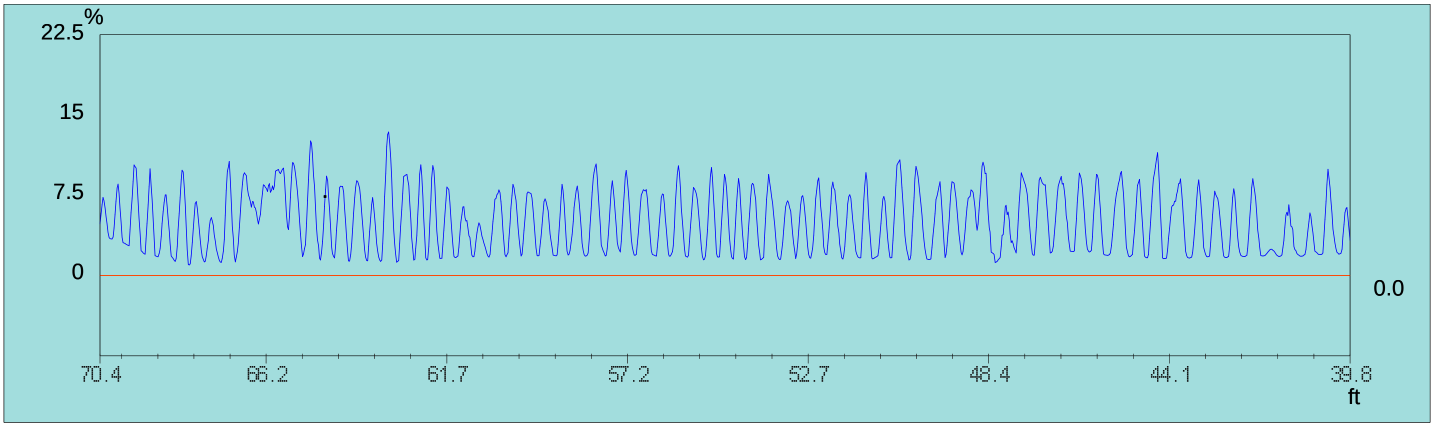
| | | | | | |
|---|---|---|---|---|---|
| Site ID | Auburn | Asset No. | 231+00 | Date | 8/9/2016 |
| City | Auburn AL | | | Material | HDPE |
| Start No | 231+00W | Finish No | 231+00E | Pipeline Length | 128ft |
| Location | Downstream | Location | Upstream | Internal Diameter | 36in |

90% - Fractile: 9.2%, Exceeded limits: 100.0%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



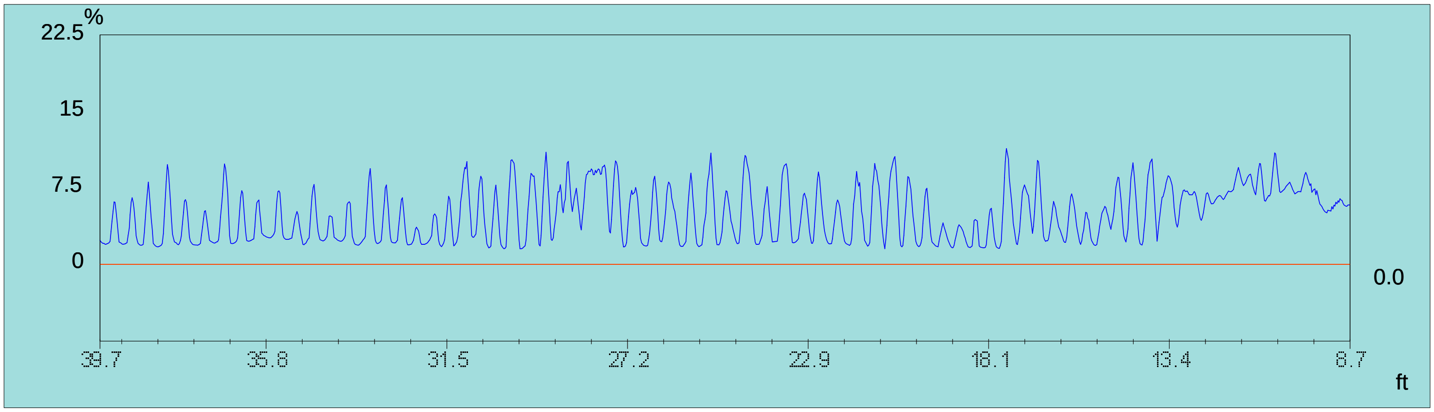




Comments

r
10.
6.7
3.3
0%

# Ovality Analysis Report



# Appendix B: Culvert Material Selection Specification

Directions: Input the predetermined site/project specific conditions into the blank column shown. Compare the inputted data to the data shown to the right of the bolded black line. Note if additional protection is required.

| | Site Conditions | Reinforced Concrete | Galvanized Steel | Aluminized Steel | Aluminum | Plastic |
|---|---|---|---|---|---|---|
| **Service Life (Years)** | | + 75 | 75 | 75 | 75 | 75 |
| **Sulfate Content (ppm)** | | ≤ 1,000 [(1)] | ≤ 1,000[(1)] | ≤ 1,000[(1)] | ≤ 1,000[(1)] | N/A |
| **Resistivity (ohm-cm)** | | > 3,000[(2)] | > 3,000[(2)] | > 3,000[(2)] | > 3,000[(2)] | N/A |
| **pH Levels** | | pH < 5 | 6 < pH < 10 | 5 < pH < 9 | 5 < pH < 9 | N/A |
| **Abrasion (Level)** | | 1, 2, 3 | 1, 2, 3[(3)] | 1, 2, 3[(3)] | 1, 2, 3[(3)] | 1, 2, 3 |
| **Ultraviolet Radiation** | | N/A | N/A | N/A | N/A | Highly Sensitive |
| **Flammability/ Heat** | | N/A | Sensitive | Sensitive | Sensitive | Highly Sensitive |
| **Max. Pipe Diameter (inch)** | | 96 | 72 | 72 | 72 | 48 |
| **Min. Fill Height (feet)** | | 1 – 2 | 1 – 2 | 1 – 2 | 1 – 2 | 2 |
| **Max. Fill Height (feet)** | | N/A | N/A | N/A | N/A | 20 |
| **Quality Controlled Installation** | | N/A | Sensitive | Sensitive | Sensitive | Highly Sensitive |

Note:
[(1)] Additional protection required if the sulfate content exceeds 1,000 ppm.
[(2)] Additional protection required if the resistivity is less than 3,000 ohm-cm. Metal culverts not recommended.
[(3)] Additional protection required for moderate abrasion conditions.

| | Reinforced Concrete | Galvanized Steel | Galvanized Aluminum | Aluminum | Plastic |
|---|---|---|---|---|---|
| **Sulfate Concentration** | | | | | |
| ≤ 1,000 ppm | X | X | X | X | X |
| > 1,000 ppm | X(1) | X(1) | X(1) | X(1) | X |
| **Resistivity** | | | | | |
| < 3,000 ohm-cm | X(2) | | | | X |
| > 3,000 ohm-cm | X | X | X | X | X |
| **pH** | | | | | |
| pH < 5 | | | | | X |
| 5 < pH < 9 | X | | X | X | X |
| 6 < pH < 10 | X | X | | | X |
| **Abrasion** | | | | | |
| Level 1 Non-Abrasive | X | X | X | X | X |
| Level 2 Low Abrasion | X | X | X | X | X |
| Level 3 Moderate Abrasion | X | X(3) | X(3) | X(3) | X |
| Level 4 Severe Abrasion | X(4) | | | | |
| **Ultraviolet Radiation** | | | | | |
| Susceptible to degradation? | | | | | X |
| **Flammability/ Heat** | | | | | |
| Susceptible to degradation? | | X(5) | X(5) | X(5) | X |
| **Maximum Pipe Diameter** | | | | | |
| 6 inch – 48 inch | X | X | X | X | X |
| 54 inch – 72 inch | X | X | X | X | |
| 78 inch – 94 inch | X | | | | |
| **Minimum Fill Height** | | | | | |
| 1 feet | X | X | X | X | |
| ≥ 2 feet | X | X | X | X | X |
| **Maximum Fill Height** | | | | | |
| ≤ 20 feet | X | X | X | X | X |
| > 20 feet | X | X | X | X | |
| **Quality Control** | | | | | |
| Will installation be monitored? | | X | X | X | X |

**Environmental / Site / Project Parameters**

| | Reinforced Concrete | Galvanized Steel | Galvanized Aluminum | Aluminum | Plastic |
|---|---|---|---|---|---|
| **Service Life** | | | | | |
| ≤ 75 years | X | X | X | X | X |
| > 75 years | X | | | | |
| **Sulfate Concentration** | | | | | |
| ≤ 1,000 ppm | X | X | X | X | X |
| > 1,000 ppm | X(1) | X(1) | X(1) | X(1) | X |
| **Resistivity** | | | | | |
| < 3,000 ohm-cm | X(2) | | | | X |
| > 3,000 ohm-cm | X | X | X | X | X |
| **pH** | | | | | |
| pH < 5 | | | | | X |
| 5 < pH < 9 | X | | X | X | X |
| 6 < pH < 10 | X | X | | | X |
| **Abrasion** | | | | | |
| Level 1 Non-Abrasive | X | X | X | X | X |
| Level 2 Low Abrasion | X | X | X | X | X |
| Level 3 Moderate Abrasion | X | | | | X |
| Level 4 Severe Abrasion | X(3) | | | | |
| **Ultraviolet Radiation** | | | | | |
| Susceptible to degradation? | | | | | X |
| **Flammability/ Heat** | | | | | |
| Susceptible to degradation? | | X(4) | X(4) | X(4) | X |

Notes:

(1) Additional protection required if the sulfate content exceeds 1,000 ppm.

(2) Additional protection required if the resistivity is less than 3,000 ohm-cm. Metal culverts not recommended.

(3) Additional protection required for moderate abrasion conditions.

(4) Additional protection required for severe abrasion conditions.

(5) Protective coatings may be flammable.

**Site / Project Installation Parameters**

| | Reinforced Concrete | Galvanized Steel | Galvanized Aluminum | Aluminum | Plastic |
|---|---|---|---|---|---|
| **Maximum Pipe Diameter** | | | | | |
| 6 inch – 48 inch | X | X | X | X | X |
| 54 inch – 72 inch | X | X | X | X | |
| 78 inch – 94 inch | X | | | | |
| **Minimum Fill Height** | | | | | |
| 1 feet | X | X | X | X | |
| ≥ 2 feet | X | X | X | X | X |
| **Maximum Fill Height** | | | | | |
| ≤ 20 feet | X | X | X | X | X |
| > 20 feet | X | X | X | X | |
| **Quality Control** | | | | | |
| Will installation be monitored? | | X | X | X | X |

## Reference Information

This section provides a brief description of the key decision factors affecting the selection of culvert materials for cross-drainage application. This section shall be used in conjunction with the checklists.

### I. Sulfate Concentration

While a high concentration of sulfates may corrode metal culverts, sulfates are typically more damaging to concrete culverts. A site shall be considered corrosive if it contains ≥ 1,000 ppm.

- If the sulfate concentration is < 1,000 ppm, concrete, metal, and plastic are all acceptable culvert materials.
- If the sulfate concentration is ≥ 1,000 ppm, concrete and metal culvert materials are the most susceptible to degradation and will require additional protection.

Protective Measures

The most efficient way to protect against a high concentration of sulfates is to choose a cement with a limited amount of tricalcium aluminate. Other resistance factors may include reducing the water-to-cement ratio, using a higher strength concrete, or applying special coatings.

### II. Resistivity

Low resistivity values are typically more damaging to metal culverts. A site shall be considered corrosive if it contains ≤ 3,000 ohm-cm.

- If the resistivity value is > 3,000 ohm-cm, concrete, metal, and plastic are all acceptable culvert materials.
- If the resistivity value is ≤ 3,000 ohm-cm, metal culvert materials are the most susceptible to degradation and will require additional protection.

Metal culverts are not recommended if the resistivity value is ≤ 3,000 ohm-cm. Special coatings and/or internal and external cathodic protection is required.

### III. Hydrogen Ion Concentration (pH)

Concrete culverts and metal culverts are the most affected by pH levels. Protective coatings may be applied. However, choosing an alternative culvert material more suited for site pH levels may be more desirable.

Concrete Culverts – Concrete should not be used where the pH is less than 5. Concrete culverts are sensitive to saltwater.

Metal Culverts – Galvanized steel should not be used where the pH is outside the range of 6.0 to 10. Aluminized steel is vulnerable to alkalis and should not be used where the pH is greater than 9. Aluminized steel has an advantage over galvanized steel in lower pH environments

Plastic Culverts – High density polyethylene, polyvinyl chloride and polypropylene are not affected by pH.

### IV. Abrasion

Abrasion is typically more damaging to concrete culverts and metal culverts. Abrasion is dependent on the velocity of water. Table B-2 provides four abrasion levels characterized by the Federal Lands Highway Division.

Concrete Culverts

- Concrete is recommended for abrasive conditions.
- Protective measures shall be taken for severe abrasive conditions.

Metal Culverts

- Steel and plain aluminum are not recommended for abrasive conditions.
- Non-metallic coatings should be chosen over metallic coatings for increased abrasion resistance.

Plastic Culverts

- Plastic pipes are not recommended for severe abrasive conditions.

## V. Ultraviolent Radiation

Prolonged exposure to ultraviolet radiation is typically more damaging to plastic pipe culverts. Extra care shall be taken during the storage period. Extra care shall be taken before backfilling.

## VI. Flammability

All culvert materials are affected by fire and extremely high temperatures.

## VII. Minimum Fill Height

Concrete culverts and metal culverts typically have a minimum fill height of 12 inches to 24 inches. High density polyethylene culverts and polypropylene culverts shall have a minimum fill height of 24 inches.

## VIII. Maximum Burial Depth

<u>Concrete Culverts</u>

The burial depth of concrete culverts is dependent upon the class of pipe and the pipe diameter. Class V requires the greatest amount of cover. The burial depth increases as the pipe diameter increases. Therefore, large diameter concrete culverts are used for deep installation.

<u>Metal Culverts</u>

The burial depth for steel culverts and aluminum culverts is dependent upon pipe diameter, specified sheet thickness, and corrugations.

The burial depth increases as the specified thickness increases, but decreases the pipe diameter increases. Therefore, thicker steel and aluminum culverts with a smaller diameter are used for deep installation.

<u>Plastic Culverts</u>

The burial depth for plastic pipe culverts is dependent upon pipe material and pipe diameter. The burial depth decreases as the pipe diameter increases. Therefore, small diameter plastic culverts are used for deep installation. Plastic culverts shall not be used in deep installations where the burial depth exceeds 20 feet.

# Appendix C: Culvert Selection Based on Average Daily Traffic Flows

## Culvert Selection Based on Average Daily Traffic Flows

There are several state departments of transportation that classify the pipe material type based on recorded average daily traffic flow counts. The average daily traffic is an approximate measure of the volume of vehicular traffic on a highway or roadway. This information is especially important in transportation planning and transportation engineering. According to the Mississippi Department of Transportation's (MDOT 2007) *Pipe Culvert Material Design Criteria*, cross drain culverts must have a design life of 50 years, and alternative material types may be selected based on roadway conditions. These alternative materials are shown in Table C-1. The Missouri Department of Transportation (MoDOT) also allows alternative material types for certain roadway groups. These alternatives are shown in Table C-2.

**Table C-1: Cross Drain Culvert Material Design Criteria (MDOT 2007)**

| Condition | Alternative Material[1] |
|---|---|
| Rural Collectors and Local Roads:<br>Average Daily Traffic (ADT) ≤4000 *and*<br>Average Daily Truck (T) ≤ 400 *and*<br>Pipe size ≤ 48-inch dimeter | Concrete, Galvanized Steel, Bituminous Coated Galvanized Steel, Aluminized Type 2 Steel, Polymer Coated, Aluminum Alloy, High Density Polyethylene, Polyvinyl Chloride |
| All other functional classifications or other Collectors and Load Roads, Urban or Rural, where ADT and/or T and/or pipe size exceeds the limits shown above | Concrete only |

[1] All alternatives shall have concrete end sections

**Table C-2: Permissible Pipe Types by Group (MoDOT 2016)**

| Group A (ADT > 3,500) | Group B (ADT ≤ 3,500) | Group C (ADT < 1,700) |
|---|---|---|
| Reinforced Concrete | Group A Pipe | Group A Pipe |
| Vitrified Clay | Polypropylene | Group B Pipe |

| | | |
|---|---|---|
| Aluminum Coated Steel | High Density Polyethylene | Zinc-Coated Steel |
| Polymer Coated Steel | Steel Reinforced Polyethylene | Bituminous-Coated Steel |
| Aluminum Alloy | Polyvinyl Chloride | |
| Polypropylene ≤ 36 inch | | |
| High Density Polyethylene ≤ 24 inch | | |
| Steel Reinforced Polyethylene ≤ 24 inch | | |
| Polyvinyl Chloride ≤ 36 inch | | |

According to MoDOT's *Non-Hydraulic Considerations*,

> *For roadways with ADT ≤ 3500, corrugated polyethylene pipe (Type S) and polyvinyl chloride pipe are double walled, full circular cross section pipes, with an outer corrugated wall and a smooth inner liner. Only 12 to 60 in. diameter sizes of corrugated polyethylene, or 12 to 48 in. diameter sizes of PVC pipe, are approved for use on highway projects. Corrugations may be either annular or helical*. (MoDOT 2016)

Table C-3 shows the culvert materials and allowable sizes permitted by WisDOT based on the traffic volume range. According to the Illinois Department of Transportation's (IDOT) *Drainage Manual*, "the kind and size of culvert permitted is dependent upon location; however, 15 inch is considered the smallest size practical." Table C-4 shows the class of pipe and the minimum permissible diameter based on location. Table C-5 provides descriptions of the pipe class referenced in Table C-4. (IDOT 2011)

**Table C-3: Culvert Material Selection Criteria (WisDOT 2015)**

| Material | Design ADT | Allowable Sizes | Notes |
|---|---|---|---|
| Reinforced Concrete | Over 7,000 | 12 – 84 inches | – |
| Corrugated Steel | Under 7,000 | 12 – 84 inches | Not to be used in Corrosive Environments |
| Corrugated Aluminum | Under 1,500 | 12 – 84 inches | – |
| Corrugated Polyethylene | Under 7,000 | 12 – 36 inches | Max. Fill Height of 11 feet |
| Corrugated Polypropylene | Under 7,000 | 12 – 36 inches | Max. Fill Height of 15 feet |

**Table C-4: Culvert Material Design Criteria (IDOT 2011)**

| Type of Improvement | Kind or Class of Drainage Structure Permitted | Minimum Permissible Diameter |
|---|---|---|
| All roadways with ADT ≥ 10,000 | A | 24" – up to 200' in length<br>30" > 300' in length |
| All roadways with 4,000 ≤ ADT ≤ 10,000 | C | 18" |
| Entrances/driveway and roadways with ADT < 4,000 | D | 15" |

**Table C-5: Description of Material Class (IDOT 2011)**

| Class | Material |
|---|---|
| A | Reinforced Concrete<br>Reinforced Concrete Arch Culvert<br>Reinforced Concrete Elliptical Culvert |
| C | Reinforced Concrete<br>Reinforced Concrete Arch Culvert<br>Reinforced Concrete Elliptical Culvert<br>Polyvinyl Chloride Pipe<br>Corrugated Polyvinyl Chloride (PVC) Pipe with Smooth Interior<br>Polyvinyl Chloride (PVC) Profile Wall Pipe – 794<br>Polyvinyl Chloride (PVC) Profile Wall Pipe – 304<br>Polyethylene (PE) Pipe with a Smooth Interior<br>Polyethylene (PE) Profile Wall Pipe |

| | |
|---|---|
| | Aluminized Steel Type 2 Corrugated Culvert Pipe<br>Aluminized Steel Type 2 Corrugated Pipe Arch<br>Precoated Galvanized Corrugated Steel Culvert Pipe<br>Precoated Galvanized Corrugated Steel Pipe Arch<br>Corrugated Aluminum Alloy Pipe<br>Corrugated Aluminum Alloy Culvert Pipe Arch |
| D | Reinforced Concrete<br>Reinforced Concrete Arch Culvert<br>Reinforced Concrete Elliptical Culvert<br>Polyvinyl Chloride (PVC) Pipe<br>Corrugated Polyvinyl Chloride (PVC) Pipe with a Smooth<br>Interior Polyvinyl Chloride (PVC) Profile Wall Pipe – 794<br>Polyvinyl Chloride (PVC) Profile Wall Pipe – 304<br>Polyethylene (PE) Pipe with Smooth Interior<br>Polyethylene (PE) Profile Wall Pipe<br>Aluminized Steel Type 2 Corrugated Culvert Pipe<br>Aluminized Steel Type 2 Corrugated Pipe Arch<br>Precoated Galvanized Corrugated Steel Culvert Pipe<br>Precoated Galvanized Corrugated Steel Pipe Arch<br>Corrugated Aluminum Alloy Pipe<br>Corrugated Aluminum Alloy Culvert Pipe Arch<br>Corrugated Polyethylene (PE) Pipe with a Smooth Interior<br>Corrugated Steel Culvert Pipe<br>Corrugated Steel Pipe Arch<br>Bituminous Coated Corrugated Steel Culvert Pipe<br>Bituminous Coated Corrugated Steel Pipe Arch<br>Zinc and Aramid Fiber Composite Coated Corrugated Steel Pipe |

While the New Hampshire Department of Transportation (NHDOT) does not specifically limit culvert material types based on location, the Highway Design Manual does offers material recommendations. As stated in Chapter 6 of NHDOT's *Highway Design Manual*,

> *Plastic pipe is typically used for drainage systems parallel to the roadway, under shoulders and in ditch lines, across low volume roadways (less than 5000 ADT), and for slope drainage purposes. Reinforced concrete pipe (RCP) is typically used for all other applications. Corrugated aluminized steel pipe may be considered for use to reduce potentially high outlet velocities*. (NHDOT 2007)

Chapter 10 of the Pennsylvania Department of Transportation's (PennDOT) *Design Manual* provides a summary of acceptable criteria for alternative types of culvert pipe. This summary is shown in Table C-6. The thermoplastic pipe groups referenced are defined in Table C-7.

**Table C-6: Alternative Pipe Selection Criteria of Drainage Pipes (PennDOT 2012)**

<table>
<tr><th colspan="2">Location of Drainage Pipes</th><th colspan="2">Types of Pipes</th><th>No. of Alternates Required</th></tr>
<tr><td rowspan="4">Cross Drains under Pavement, Shoulder, or between Curbs; Parallel Storm Sewers under Pavement or between Curbs</td><td>Fill Height</td><td>Interstate/<br>Arterials</td><td>Collectors/<br>Locals</td><td rowspan="4">2</td></tr>
<tr><td>< 2 feet</td><td>100 Years Life (Pipes 1, 2, 5 and 7)</td><td>50 Years Life<br>(Pipes 1 and 3 thru 7)</td></tr>
<tr><td>2 feet – 15 feet</td><td>100 Years Life (Pipes 1, 2, 5 and 7)</td><td>50 Years Life<br>(Pipes 1 and 3 thru 7 and 8)</td></tr>
<tr><td>> 15 feet</td><td>100 Years Life (Pipes 1, 2, 5 and 7)</td><td>100 Years Life<br>(Pipes 1, 2, 5 and 7)</td></tr>
<tr><td>Parallel Storm Sewers outside of Pavement or Curbs</td><td colspan="3">50 Years Life (All pipes in LEGEND)</td><td>3</td></tr>
<tr><td>Cross Drains outside of Pavement, Shoulder or Curbs (Cross Drains in Medians, etc.)</td><td colspan="3">50 Years Life (All pipes in LEGEND except 9)</td><td>3</td></tr>
<tr><td rowspan="2">Combination Storm Sewer and Underdrain and Other Special</td><td>100 Years Life</td><td colspan="2">Pipe 2, open joint and perforated pipes 5 and 7</td><td>2</td></tr>
<tr><td>50 Years Life</td><td>Fill Height<br>< 2 feet</td><td>Pipe 3, open joint and perforated pipes 4, 5</td><td>3</td></tr>
</table>

| | | | | |
|---|---|---|---|---|
| Drainage System | | | and 7 | |
| | | Fill Height ≥ 2 feet | Pipe 3, open joint and perforated pipes 4, 5, 7 and 8 | |
| Slop Pipes | 50 Years Life (Pipes 4 thru 9) | | | 2 |
| Side Drains (Driveways, etc.) | 25 Years Life (All pipes in LEGEND) | | | 3 |

LEGEND

1. DIP = Ductile Iron Pipe.
2. RCP (Type A) = Reinforced Concrete Pipe, heavy duty.
3. RCP (Type B) = Reinforced Concrete Pipe, normal duty (48 in max)
4. CGSP = Corrugated Galvanized Steel Pipe
5. CASP = Corrugated Aluminized Steel Pipe
6. CCGSP = Coated (Polymer) Corrugated Galvanized Steel Pipe
7. CAAP = Corrugated Aluminum Alloy Pipe
8. TP (Group I, II, III, IV or VI) = Thermoplastic Pipe, Group I, II, III, IV or VI (60 in max)
9. TP (Group V) = Thermoplastic Pipe, Group V - Corrugated Polyethylene (36 in max)

**Table C-7: Thermoplastic Pipe Groups (PennDOT 2016)**

| Group Number | Material | Max. Fill Height | Min. Cover |
|---|---|---|---|
| I | Polyethylene, Polyvinyl Chloride | 15 feet | 1.50 feet |
| II | Polyethylene | 12 feet | 1.50 feet |
| III | Polyethylene, Polyvinyl Chloride | 8 feet | 2.00 feet |
| IV | Polyethylene | 7 feet | 2.50 feet |
| V | Polyethylene | 7 feet | 2.50 feet |
| VI | Polyethylene, Polypropylene | 15 feet | 2.00 feet |

| | Sulfates | Resistivity | pH Level | Abrasion | Max. Diameter | Min. Fill Height | Max. Fill Height | UV, Fire | Material |
|---|---|---|---|---|---|---|---|---|---|
| > 4,000 ADT | ≥ 1,000 | ≤ 3,000 | | | | | | | Note 1 |
| | < 1,000 | > 3,000 | pH < 5 | 1 – 3 | 48 | 2 | 20 | No | Note 2 |
| | | | | 1 – 3 | 96 | 1 – 2 | – | Yes | Note 2 |
| | | | | 1 – 2 | 72 | 1 – 2 | – | Yes | Note 2 |
| | < 1,000 | > 3,000 | 6 ≤ pH < 10 | 1 – 3 | 48 | 2 | 20 | No | RCP |
| | | | | 1 – 3 | 96 | 1 – 2 | – | Yes | RCP |
| | | | | 1 – 2 | 72 | 1 – 2 | – | Yes | RCP |
| | < 1,000 | > 3,000 | 5 ≤ pH < 9 | 1 – 3 | 48 | 2 | 20 | No | RCP |
| | | | | 1 – 3 | 96 | 1 – 2 | – | Yes | RCP |
| | | | | 1 – 2 | 72 | 1 – 2 | – | Yes | RCP |
| ≤ 4,000 ADT | ≥ 1,000 | ≤ 3,000 | | | | | | | Note 1 |
| | < 1,000 | > 3,000 | pH < 5 | 1 – 3 | 48 | 2 | 20 | No | HDPE, PP, PVC |
| | | | | 1 – 3 | 96 | 1 – 2 | – | Yes | Note 3 |
| | | | | 1 – 2 | 72 | 1 – 2 | – | Yes | Note 3 |
| | < 1,000 | > 3,000 | 6 ≤ pH < 10 | 1 – 3 | 48 | 2 | 20 | No | HDPE, PP, PVC, RCP, CSP |
| | | | | 1 – 3 | 96 | 1 – 2 | – | Yes | RCP |
| | | | | 1 – 2 | 72 | 1 – 2 | – | Yes | RCP, CSP |
| | < 1,000 | > 3,000 | 5 ≤ pH < 9 | 1 – 3 | 48 | 2 | 20 | No | HDPE, PP, PVC, RCP, CAP |
| | | | | 1 – 3 | 96 | 1 – 2 | – | Yes | RCP |
| | | | | 1 – 2 | 72 | 1 – 2 | – | Yes | RCP, CAP |

1 Some Departments of Transportation allow plastic pipe be used on roadways with an ADT < 7,000. However, most Departments of Transportation restrict the use of plastic pipe on roadways with an ADT < 5,000 or an ADT < 3,500. Therefore, if the site is corrosion and the ADT > 4,000 then a reinforced concrete culvert with increased corrosion protection is recommended.
2 A pH level less than 5 is extremely acidic, and a plastic pipe is recommended. However, see Note 1.

3 A pH level less than 5 is extremely acidic, and a plastic pipe is recommended. However, the diameter and fill height is conservatively limited for plastic pipe, and it is not recommended plastic pipe be installed at a site with long term sunlight exposure or a high likelihood of fire. Therefore, a reinforced concrete culvert with increased corrosion protection is recommended.

4 Culvert Material acronyms are as defined: RCP – Reinforced concrete pipe, RSP – Corrugated steel pipe, CAP – Corrugated aluminum pipe, HDPE – High density polyethylene, PP – Polypropylene, PVC – Polyvinyl chloride.

| | | Reinforced Concrete | Corrugated Steel | Corrugated Aluminum | High Density Polyethylene | Polypropylene | Polyvinyl Chloride |
|---|---|---|---|---|---|---|---|
| **Durability** | | | | | | | |
| Sulfates | Ppm | < 1,000 | < 1,000 | < 1,000 | No limit | No limit | No limit |
| Resistivity | Ohm-cm | > 3,000 | > 3,000 | > 3,000 | No limit | No limit | No limit |
| pH Level | – | pH > 5 | 6 < pH < 10 | 5 < pH < 9 | No limit | No limit | No limit |
| Abrasion | – | 1, 2, 3 | 1, 2 | 1, 2 | 1, 2, 3 | 1, 2, 3 | 1, 2, 3 |
| **Roadway/Site** | | | | | | | |
| ADT | – | No limit | < 4,000 | < 4,000 | < 4,000 | < 4,000 | < 4,000 |
| Max. Diameter | Inch | 96 | 72 | 72 | 48 | 48 | 48 |
| Min. Fill Height | Feet | 1 – 2 | 1 – 2 | 1 – 2 | 2 | 2 | 2 |
| Max. Fill Height | Feet | No limit | No limit | No limit | 20 | 20 | 20 |
| **Other** | | | | | | | |
| Service Life | Years | > 75 | ≤ 75 | ≤ 75 | ≤ 75 | ≤ 75 | ≤ 75 |
| UV Exposure | – | No Concern | No Concern | No Concern | Harmful | Harmful | Harmful |
| Flammability | – | No Concern | Minor Concern | Minor Concern | Harmful | Harmful | Harmful |

# Appendix D: Design and Selection of Pipes Guidance

## Design and Selection of Pipes Guidance

In 2011, Kevin White of E.L. Robinson Engineering published the report *Guidance for Design and Selection of Pipes* for the Technical Committee on Hydrology and Hydraulics. The report was funded by the National Cooperative Highway Research Program. One of the main objectives of the report was to develop a protocol for the selection of pipe culvert materials. The protocol was based on engineering and economic considerations. According to Mr. White, there are four critical components that must be considered to have a successful culvert management process. These four components include culvert material selection, culvert installation, short-term monitoring, and long-term monitoring.

Mr. White performed an extensive literature review to gather information pertaining to material selection processes, durability, joint performance, hydraulic design, environmental considerations, and structural design. Some of the information obtained from the literature review is shown in Table D-1. The information has been extracted verbatim from Mr. White's report and tabulated for clarity.

**Table D-1: Summary of Literature Review (White and Hurd 2011)**

| Pipe Material | Literature Review |
|---|---|
| Aluminum Coated Corrugated Steel Pipe | Type-2, aluminized-coated corrugated steel pipe provided a significantly longer service life (anywhere from 3 to 8 times) than galvanized corrugated steel pipe under most conditions |
| Polymer Coated Corrugated Steel Pipe | Since most of the protective coatings previously studied are either no longer available or no longer recommended, polymer coated corrugated steel pipe is now the most commonly recommended protection |
| Corrugated Aluminum Pipe | The general consensus among many states is that |

| | |
|---|---|
| | corrugated aluminum pipe will provide the desired design service life if pH is roughly between 5.5 and 8.5, resistivity above 1500 ohm-cm, and the site is not too abrasive |
| Concrete Pipe | Similar to corrugated aluminum pipe, the general consensus among many states is that concrete pipe will provide the desired service life if pH is roughly between 5.0 and 9.0 |
| Plastic Pipe | It is generally recognized that plastic pipe is resistant to corrosion caused by most naturally occurring chemicals found in wide ranging soil types, and flow pH and resistivity. The major question as to the long-term durability of plastic pipes is in regards to resistance to slow crack growth and oxygen degradation |

Mr. White focused a portion of his literature review to State Department's specifications and policies. It was discovered that while some states based material selection solely on service life requirements, most based pipe material according to roadway functional classification and/or average daily traffic counts. Metal pipes and plastic pipes received the greatest restrictions, and the cover height for plastic pipes varied widely among states.

> Cover height requirements for concrete and metal pipe were generally consistent. However, those for plastic pipe varied widely among the different states. Minimum cover ranged from 1 foot to 3 feet and maximum cover ranged from less than 10 feet to 50 feet (White and Hurd 2011).

A section of *Guidance for Design and Selection of Pipes* is devoted to pointing out current gaps in knowledge of insufficient information needed to provide meaningful solution strategies. A few of these current knowledge gaps are shown below as directly stated in the report.

Culvert Material Selection

1. Definitions of service life for the material service life predictive models developed for different pipe materials are inconsistent.

2. There is no consensus among the state DOTs of what the particular design service life should be for any particular situation.

3. The roughness coefficients, Manning's "n" values, used in the hydraulic design of fish passage culverts with "natural" stream bottoms should be verified

Culvert Installation

1. There is a lack of guidance for the direct measurement of pipe embedment soil stiffness to supplement and eventually replace the use of density measurements.

2. A consistent manner for determining trench width requirements for all pipe types is necessary.

Short-term Monitoring

1. There still exists a wide disparity among state DOT's regarding post construction inspection, appraisal, and acceptance requirements for different pipe materials.

2. There is also a difference of opinion on the level of "unacceptable" performance which would require repair, replacement, or acceptance with penalty (deduction from the bid price).

Long-term Monitoring

1. There is little consistency among state DOT's and their district offices on development and management of a culvert inventory.

2. There is a critical need for a comprehensive, state-of-the-art computerized tool that may be used by DOT's for documenting and managing culvert and storm drain facilities once they are identified, evaluated, and rated.

According to the report,

> The "Alternate Pipe Material Selection Protocol" presented in this document is based on the premise that all common pipe materials with recognized national material and design specifications are acceptable until they fail to meet the desired performance requirements for any of the following five design criteria: Design Service Life (Durability), Structural Design, Hydraulic Design, Joint Performance, and Environmental Considerations.

> It is not the intention of the protocol to dictate what the specific performance requirements for each criterion should be, but rather to implement a technically sound selection (elimination) process (2011).

The "Alternative Pipe Material Selection Protocol" is divided into three phases. Phase I determines the desired pipe performance based on site specific conditions. Phase II determines the hydraulic and structural design for those pipe material that have not been eliminated in Phase I. Phase III contains additional elimination steps based on the hydraulic and structural design of Phase II. The "Alternative Pipe Material Selection Protocol" is shown in Figure D-1.

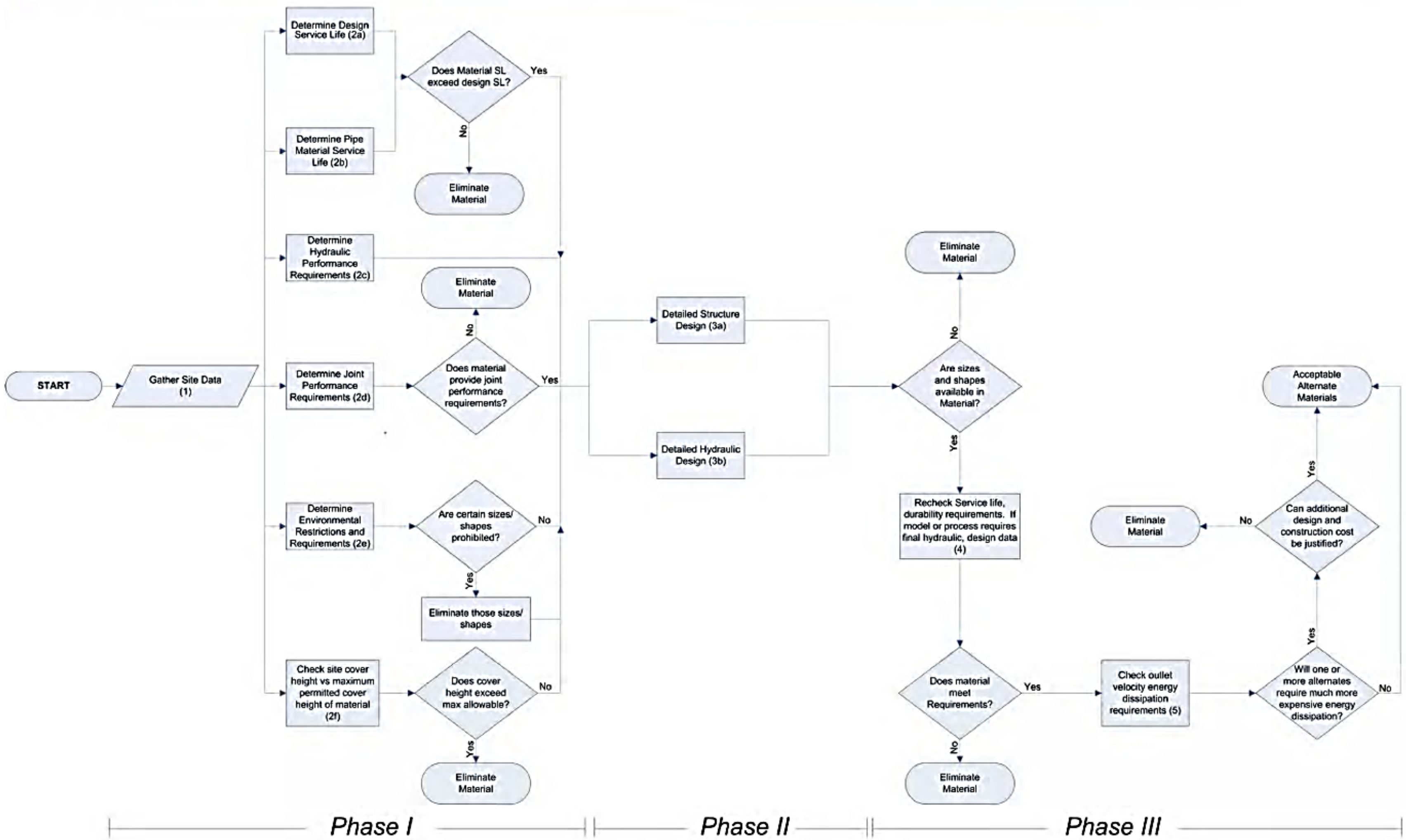



**Figure D-1: The "Alternative Pipe Material Selection Protocol" (White and Hurd 2011)**

# Appendix E: Jeyapalan Results Comparison

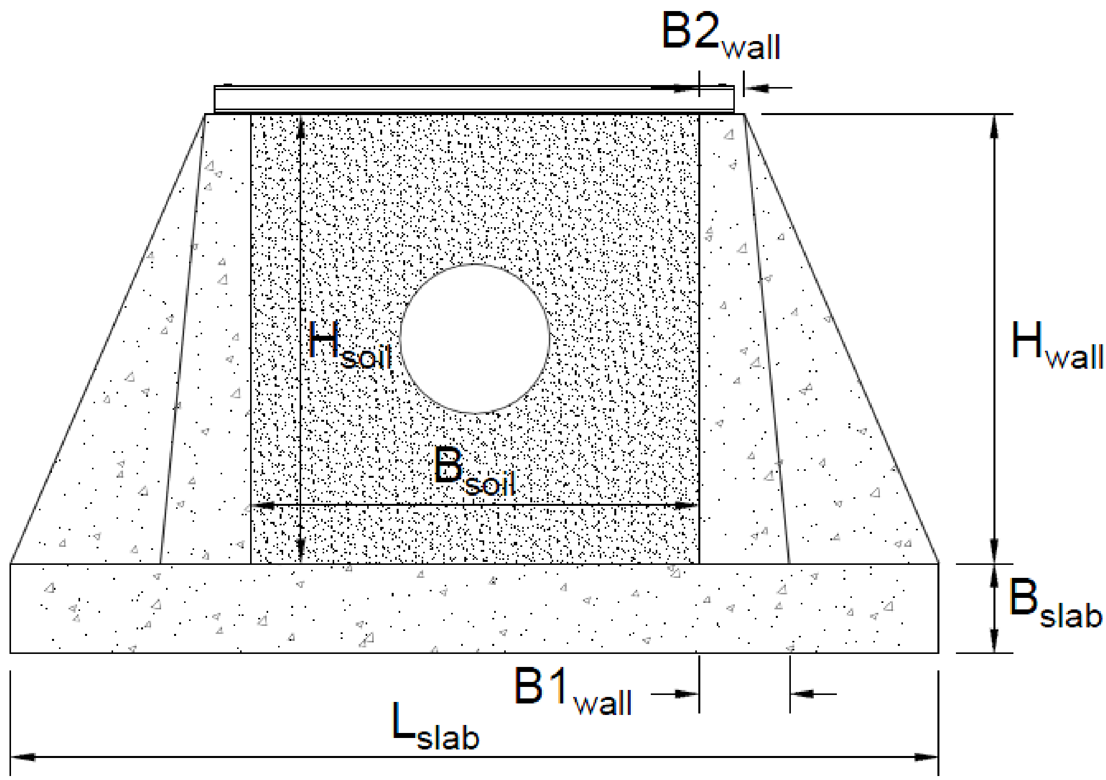
## Jeyapalan and Watkins (2004) Modulus of Soil Reaction Comparison

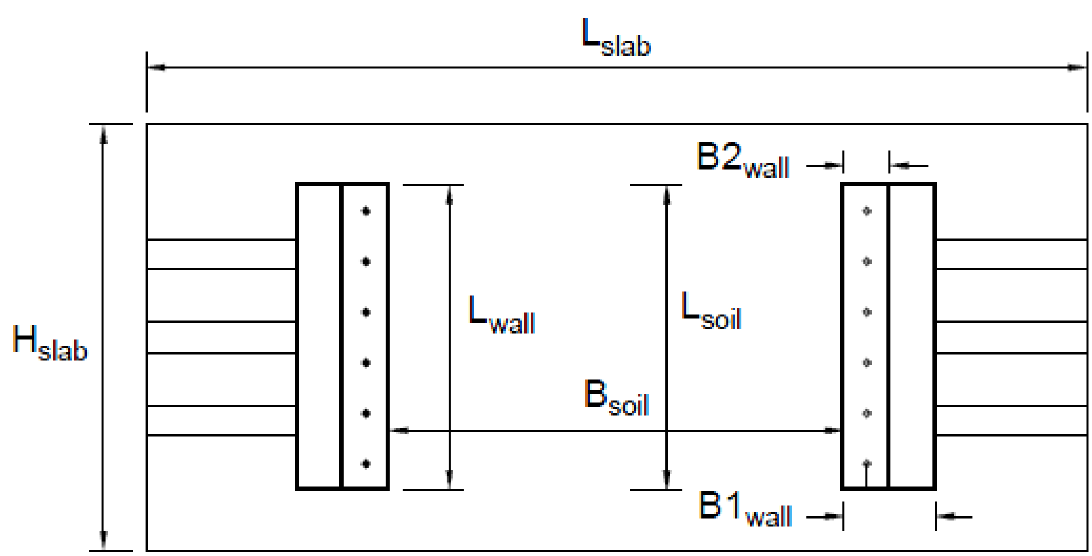
| Site no. | Pipe type | Diameter (in.) | $EI/R^3$ (psi) | Soil type | Degree of compaction | Howard's $E'$ (psi) | Fill height (ft) | Fill density (pcf) | Predicted $dx/D$(%) | Actual $dx/D$(%) | $E'$ recalculated | $E'$ % error |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | CMP | 24 | 37.5 | III | Slight | 400 | 33.1 | 105 | 3.9 | 3.2 | 622 | −55 |
| 2 | CMP | 42 | 10.3 | III | Slight | 400 | 33.5 | 105 | 7.1 | 6.6 | 438 | −9 |
| 3 | CI | 42 | 160.9 | III | Slight | 400 | 34.2 | 105 | 1.3 | 0.6 | 4,176 | −944 |
| 4 | CMP | 42 | 10.3 | III | Slight | 400 | 34.9 | 105 | 7.4 | 6.5 | 473 | −18 |
| 5 | SI | 48 | 9.3 | III | Slight | 400 | 27.9 | 105 | 6.1 | 6.2 | 385 | 4 |
| 6 | CMP | 30 | 0.9 | V | Slight | 1,000 | 12.0 | 107 | 1.4 | 2.1 | 681 | 32 |
| 7 | SI | 30 | 27.1 | V | Slight | 1,000 | 12.0 | 107 | 1.0 | 1.0 | 1,017 | −2 |
| 8 | CI | 30 | 32.6 | V | Slight | 1,000 | 12.0 | 107 | 1.0 | 0.8 | 1,293 | −29 |
| 9 | SI | 30 | 229.3 | V | Slight | 1,000 | 12.0 | 107 | 0.3 | 0.3 | 1,113 | −11 |
| 10 | CMP | 20 | 1.0 | V | Slight | 1,000 | 12.0 | 107 | 1.4 | 2.5 | 568 | 43 |
| 11 | CMP | 20 | 65.5 | V | Slight | 1,000 | 12.0 | 107 | 0.7 | 1.0 | 388 | 61 |
| 12 | CMP | 42 | 16.6 | IV | Slight | 400 | 15.0 | 121 | 3.1 | 3.2 | 374 | 7 |
| 13 | CMP | 42 | 13.2 | IV | Moderate | 1,000 | 16.0 | 130 | 1.9 | 1.8 | 1,099 | −10 |
| 14 | CMP | 36 | 8.8 | IV | Moderate | 1,000 | 15.0 | 121 | 1.8 | 1.8 | 1,004 | 0 |
| 15 | CMP | 36 | 8.8 | IV | Slight | 400 | 15.0 | 121 | 3.8 | 3.5 | 446 | −12 |
| 16 | CMP | 42 | 7.0 | IV | Moderate | 1,000 | 15.0 | 121 | 1.9 | 1.8 | 1,033 | −3 |
| 17 | CMP | 42 | 7.0 | IV | Slight | 400 | 15.0 | 121 | 4.0 | 3.2 | 531 | −33 |
| 18 | CMP | 48 | 4.8 | IV | Moderate | 1,000 | 15.0 | 121 | 1.9 | 1.8 | 1,069 | −7 |
| 19 | CMP | 48 | 4.8 | IV | Slight | 400 | 15.0 | 121 | 4.3 | 4.3 | 402 | 0 |
| 20 | CMP | 60 | 3.5 | IV | Moderate | 1,000 | 15.0 | 121 | 2.0 | 1.6 | 1,234 | −23 |
| 21 | CMP | 60 | 3.5 | IV | Slight | 400 | 15.0 | 121 | 4.5 | 2.9 | 655 | −64 |
| 22 | CMP | 180 | 6.8 | IV | Moderate | 1,000 | 42.0 | 120 | 5.2 | 5.3 | 971 | 3 |
| 23 | CMP | 180 | 3.2 | V | Moderate | 2,000 | 41.5 | 120 | 2.8 | 3.7 | 1,480 | 26 |
| 24 | CMP | 120 | | IV | Moderate | 1,000 | 13.0 | 110 | 1.6 | 4.0 | 407 | 59 |
| 25 | DI | 36 | 29.0 | II | Dump | 50 | 5.0 | 94 | 1.0 | 0.6 | 416 | −733 |
| 26 | AL CMP | 40 | 7.0 | III | High | 2,000 | 6.0 | 130 | 0.4 | 0.3 | 2,845 | −42 |
| 27 | S | 34 | 35.0 | IV | High | 2,000 | 6.0 | 120 | 0.3 | 0.3 | 2,158 | −8 |
| 28 | S | 34 | 35.0 | IV | High | 2,000 | 8.5 | 120 | 0.4 | 0.4 | 2,329 | −16 |
| 29 | S | 72 | 6.7 | V | Moderate | 2,000 | 4.4 | 111 | 0.3 | 0.1 | 5,450 | −173 |
| 30 | R | 30 | 2.0 | II | Dump | 50 | 4.5 | 115 | 7.1 | 7.8 | 43 | 15 |
| 31 | R | 30 | 2.0 | II | Slight | 200 | 4.5 | 115 | 2.5 | 3.5 | 136 | 32 |
| 32 | R | 30 | 2.0 | II | High | 1,000 | 4.5 | 115 | 0.6 | 0.1 | 5,859 | −486 |
| 33 | R | 30 | 2.0 | V | Dump | 200 | 4.5 | 115 | 2.6 | 5.1 | 83 | 59 |
| 34 | R | 30 | 2.0 | V | Slight | 1,000 | 4.5 | 115 | 0.6 | 0.6 | 949 | 5 |
| 35 | PVC | 12 | | V | Slight | 1,000 | 2.5 | 124 | 0.4 | 0.8 | 441 | 56 |
| 36 | S | 60 | 4.9 | II | High | 1,000 | 5.2 | 110 | 0.5 | 0.3 | 2,090 | −109 |
| 37 | S | 60 | 4.9 | V | High | 3,000 | 6.8 | 110 | 0.2 | 0.1 | 8,435 | −181 |
| 38 | S | 60 | 4.9 | V | High | 3,000 | 9.5 | 110 | 0.5 | 0.3 | 3,885 | −30 |
| 39 | S | 60 | 11.6 | V | High | 3,000 | 9.0 | 110 | 0.4 | 0.05 | 22,351 | −645 |
| 40 | S | 60 | 4.9 | V | High | 3,000 | 11.7 | 110 | 0.5 | 0.5 | 2,850 | 5 |
| 41 | S | 60 | 7.8 | V | High | 3,000 | 13.8 | 110 | 0.6 | 0.5 | 3,328 | −11 |
| 42 | S | 60 | 7.8 | V | High | 3,000 | 15.3 | 110 | 0.6 | 0.7 | 2,609 | 13 |
| 43 | S | 90 | 2.3 | II | High | 1,000 | 5.0 | 110 | 0.6 | 0.2 | 3,093 | −209 |
| 44 | S | 90 | 2.3 | V | High | 3,000 | 6.5 | 110 | 0.3 | 0.2 | 4,032 | −34 |
| 45 | S | 90 | 2.3 | V | High | 3,000 | 9.5 | 110 | 0.4 | 0.4 | 2,936 | 2 |
| 46 | S | 90 | 2.3 | V | High | 3,000 | 14.0 | 110 | 0.6 | 0.2 | 8,728 | −191 |
| 47 | S | 90 | 3.4 | V | High | 3,000 | 40.0 | 110 | 1.6 | 1.2 | 4,119 | −37 |
| 48 | S | 34 | 186.0 | III | High | 2,000 | 5.6 | 120 | 0.2 | 0.7 | −1,956 | 198 |
| 49 | S | 34 | 186.0 | III | High | 2,000 | 5.5 | 120 | 0.2 | −0.9 | −3,884 | 294 |
| 50 | S | 42 | 171.0 | V | High | 3,000 | 10.0 | 120 | 0.2 | 0.1 | 10,858 | −262 |
| 51 | S | 126 | 2.1 | IV | High | 2,000 | 10.0 | 120 | 0.7 | 0.6 | 2,242 | −12 |
| 52 | S | 126 | 2.1 | IV | Moderate | 1,000 | 10.0 | 120 | 1.3 | 1.6 | 819 | 18 |
| 53 | R | 24 | 7.0 | IV | Slight | 400 | 19.0 | 120 | 5.0 | 3.1 | 723 | −81 |
| 54 | R | 24 | 7.0 | IV | High | 2,000 | 18.0 | 120 | 1.2 | 1.1 | 2,121 | −6 |
| 55 | R | 24 | 3.0 | IV | High | 2,000 | 17.5 | 120 | 1.1 | 0.3 | 7,920 | −296 |
| 55a | R | 24 | 7.0 | IV | High | 2,000 | 17.5 | 120 | 1.1 | 0.3 | 7,854 | −293 |
| 56 | R | 24 | 3.0 | VI | Compacted | 3,000 | 17.0 | 120 | 0.7 | −0.2 | −11,661 | 489 |
| 56a | R | 24 | 7.0 | VI | Compacted | 3,000 | 17.0 | 120 | 0.7 | −0.2 | −11,727 | 491 |
| 57 | R | 24 | 3.0 | VI | Compacted | 3,000 | 17.0 | 120 | 0.7 | −0.3 | −7,791 | 360 |
| 57a | R | 24 | 7.0 | VI | Compacted | 3,000 | 17.0 | 120 | 0.7 | −0.3 | −7,856 | 362 |
| 58 | R | 24 | 3.0 | IV | High | 2,000 | 16.0 | 120 | 1.0 | 0.2 | 10,880 | −444 |

| Site no. | Pipe type | Diameter (in.) | $EI/R^3$ (psi) | Soil type | Degree of compaction | Howard's $E'$ (psi) | Fill height (ft) | Fill density (pcf) | Predicted $dx/D$(%) | Actual $dx/D$(%) | $E'$ recalculated | $E'$ % error |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 58a | R | 24 | 7.0 | IV | High | 2,000 | 16.0 | 120 | 1.0 | 0.2 | 10,814 | −441 |
| 59 | R | 24 | 3.0 | VI | Compacted | 3,000 | 15.0 | 120 | 0.7 | 0.7 | 2,878 | 4 |
| 60 | R | 24 | 3.0 | VI | Compacted | 3,000 | 14.0 | 120 | 0.6 | 0.4 | 4,732 | −58 |
| 61 | R | 24 | 3.0 | VI | Compacted | 3,000 | 13.0 | 120 | 0.6 | 0.7 | 2,488 | 17 |
| 62 | R | 24 | 3.8 | III | Moderate | 1,000 | 8.0 | 105 | 0.9 | 0.6 | 1,532 | −53 |
| 63 | R | 24 | 2.6 | III | Moderate | 1,000 | 8.0 | 102 | 0.9 | 0.7 | 1,284 | −28 |
| 64 | R | 24 | 3.8 | II | Slight | 200 | 8.0 | 110 | 3.8 | 2.5 | 338 | −69 |
| 65 | R | 24 | 2.6 | II | Slight | 200 | 8.0 | 106 | 4.0 | 3.2 | 259 | −30 |
| 66 | R | 24 | 3.8 | III | High | 2,000 | 18.0 | 103 | 1.0 | 0.3 | 6,973 | −249 |
| 67 | R | 24 | 2.6 | III | High | 2,000 | 18.0 | 102 | 1.0 | 0.7 | 2,943 | −47 |
| 68 | R | 24 | 2.6 | II | Slight | 200 | 18.0 | 101 | 8.5 | 7.0 | 253 | −27 |
| 69 | R | 24 | 3.8 | II | Slight | 200 | 18.0 | 103 | 8.0 | 7.6 | 215 | −8 |
| 70 | R | 39 | 1.6 | V | Slight | 1,000 | 4.0 | 122 | 0.8 | 0.7 | 767 | 23 |
| 71 | R | 48 | 2.0 | IV | High | 2,000 | 15.0 | 120 | 1.0 | 1.1 | 1,830 | 8 |
| 72 | R | 48 | 2.0 | V | High | 3,000 | 15.0 | 120 | 0.7 | 0.8 | 2,529 | 16 |
| 73 | S | 48 | 1.2 | IV | High | 2,000 | 15.0 | 120 | 1.0 | 1.1 | 1,843 | 8 |
| 74 | S | 48 | 1.2 | V | High | 3,000 | 15.0 | 120 | 0.7 | 0.7 | 2,908 | 3 |
| 75 | PT | 48 | 5.7 | IV | High | 2,000 | 15.0 | 120 | 1.0 | 1.1 | 1,769 | 12 |
| 76 | PT | 48 | 5.7 | V | High | 3,000 | 15.0 | 120 | 0.7 | 0.6 | 3,322 | −11 |
| 77 | S | 24 | 13.0 | II | Dump | 50 | 11.0 | 83 | 4.0 | 4.3 | 29 | 43 |
| 78 | S | 24 | 13.0 | II | Slight | 200 | 11.0 | 83 | 2.5 | 1.6 | 437 | −118 |
| 79 | S | 24 | 13.0 | IV | Dump | 100 | 11.0 | 83 | 3.3 | 2.3 | 239 | −139 |
| 80 | S | 24 | 13.0 | IV | Moderate | 1,000 | 11.0 | 83 | 0.9 | 0.3 | 3,252 | −225 |
| 81 | S | 24 | 13.0 | IV | High | 2,000 | 11.0 | 83 | 0.3 | 0.1 | 10,181 | −409 |
| 82 | S | 16 | 21.0 | IV | Dump | 100 | 11.0 | 83 | 2.3 | 1.8 | 233 | −133 |
| 83 | S | 16 | 21.0 | IV | Moderate | 1,000 | 11.0 | 83 | 0.8 | 0.6 | 1,388 | −39 |
| 84 | S | 30 | 7.0 | IV | Dump | 100 | 11.0 | 83 | 4.8 | 2.9 | 244 | −144 |
| 85 | S | 30 | 7.0 | IV | Slight | 400 | 11.0 | 83 | 2.0 | 2.2 | 358 | 11 |
| 86 | S | 36 | 11.0 | IV | Dump | 100 | 11.0 | 83 | 3.7 | 3.8 | 93 | 7 |
| 87 | S | 36 | 11.0 | IV | Slight | 400 | 11.0 | 83 | 1.8 | 2.2 | 292 | 27 |
| 88 | S | 24 | 13.0 | IV | Dump | 100 | 8.0 | 83 | 2.4 | 1.8 | 207 | −107 |
| 89 | R | 42 | 20.0 | VI | Compacted | 3,000 | 6.0 | 125 | 0.3 | −0.3 | −3,174 | 206 |
| 90 | R | 42 | 20.0 | VI | Dump | 1,000 | 6.0 | 125 | 0.6 | 0.4 | 1,807 | −81 |
| 91 | PVC | 12 | 0.3 | II | Dump | 50 | 3.0 | 55 | 3.4 | 2.1 | 85 | −69 |
| 92 | PVC | 12 | 0.3 | II | Dump | 50 | 2.5 | 75 | 3.9 | 7.6 | 23 | 54 |
| 93 | PVC | 12 | 0.3 | II | Dump | 50 | 2.5 | 79 | 4.1 | 4.0 | 51 | −3 |
| 94 | PVC | 12 | 0.3 | II | Dump | 50 | 2.0 | 78 | 3.2 | 2.9 | 56 | −13 |
| 95 | PVC | 12 | 0.3 | II | Slight | 200 | 3.0 | 80 | 1.3 | 1.7 | 156 | 22 |
| 96 | PVC | 12 | 0.3 | II | High | 1,000 | 2.0 | 50 | 0.1 | 0.3 | 375 | 63 |
| 97 | PVC | 12 | 0.3 | II | Dump | 50 | 2.0 | 50 | 2.1 | 2.9 | 34 | 31 |
| 98 | R | 24 | 2.1 | III | Moderate | 1,000 | 10.0 | 89 | 1.0 | 0.9 | 1,091 | −9 |
| 99 | R | 24 | 2.1 | V | Moderate | 2,000 | 10.0 | 89 | 0.5 | 0.3 | 3,343 | −67 |
| 100 | R | 24 | 2.1 | V | Moderate | 2,000 | 10.0 | 89 | 0.5 | −0.6 | −1,723 | 186 |
| 101 | R | 24 | 2.1 | V | Moderate | 2,000 | 10.0 | 89 | 0.5 | 0.7 | 1,413 | 29 |
| 102 | R | 24 | 2.1 | V | Moderate | 2,000 | 10.0 | 89 | 0.5 | 0.5 | 1,992 | 0 |
| 103 | R | 24 | 2.1 | V | High | 3,000 | 10.0 | 89 | 0.3 | −0.1 | −10,166 | 439 |
| 104 | R | 24 | 2.1 | V | Moderate | 2,000 | 10.0 | 89 | 0.5 | 0.2 | 5,032 | −152 |
| 105 | R | 24 | 2.1 | III | Slight | 400 | 10.0 | 89 | 2.3 | 2.8 | 327 | 18 |
| 106 | R | 24 | 2.1 | III | Moderate | 1,000 | 15.0 | 89 | 1.5 | 3.6 | 388 | 61 |
| 107 | R | 24 | 2.1 | V | Slight | 1,000 | 15.0 | 89 | 1.5 | 1.0 | 1,485 | −49 |
| 108 | R | 24 | 2.1 | V | Moderate | 2,000 | 15.0 | 89 | 0.7 | 0.3 | 5,032 | −152 |
| 109 | R | 24 | 2.1 | V | Moderate | 2,000 | 15.0 | 89 | 0.7 | 0.9 | 1,654 | 17 |
| 110 | R | 24 | 2.1 | V | Moderate | 2,000 | 15.0 | 89 | 0.7 | 0.2 | 7,565 | −278 |
| 111 | R | 24 | 2.1 | V | Moderate | 2,000 | 15.0 | 89 | 0.7 | 0.8 | 1,865 | 7 |
| 112 | R | 24 | 2.1 | V | Slight | 1,000 | 15.0 | 89 | 1.5 | 3.9 | 355 | 64 |
| 113 | R | 24 | 2.1 | III | Dump | 100 | 15.0 | 89 | 11.3 | 2.9 | 490 | −390 |

Note: CMP=corrugated metal pipe, CI=cast iron pipe, SI=smooth iron pipe, AL-CMP=corrugated aluminum pipe, DI=ductile iron pipe, S=steel, R=reinforced plastic mortar pipe, and PT=pretensioned concrete pipe. Type I-fine grained soils with liquid limit (LL) >50, Type II-fine grained soils with LL<50 and fines >75%, Type III-fine grained soils with LL<50 and fines <75%, Type IV-coarse grained soils with fines >12%, Type V-coarse grained soils with fines <12%, and Type VI-crushed rock. Slight compaction≤85% Proctor or 40% relative density (RD), moderate=between 86 and 95% Proctor or 41 and 70% RD, and high≥95% Proctor or 70% RD.

# Appendix F: Calculations for Testing Apparatus Design




Dimensions

| | | |
|---|---|---|
| $H_{slab} := 14ft$ ft | $H_{wall} := 15ft$ | $H_{soil} := 15ft$ |
| $L_{slab} := 31ft$ ft | $L_{wall} := 10ft$ | $L_{soil} := 10ft$ |
| $B_{slab} := 4ft$ ft | $B1_{wall} := 4ft$ | $B_{soil} := 15ft$ |
| | $B2_{wall} := 3ft$ | |

## Preliminary Calculations for Deep Burial Test Frame

Assumptions

Soil Properties

$\gamma_{soil} := 120pcf$

Steel Member Properties

$\sigma_y := 36000psi$ psi

$Es := 29000ksi$ psi

Concrete Properties

$\gamma_{conc} := 150pcf$

$f_c := 6000psi$ psi

$$\beta_1 := 1.05 - .05 \cdot \frac{f_c}{1000psi} = 0.75$$

$$Ec := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi = 4415.201 \cdot ksi$$

$\varepsilon_{cu} := 0.003$

Strength Reduction Factors

$\phi_n := 0.90$ Tension

$\phi_a := 0.75$ Anchors

$\phi_{vc} := 0.75$ Shear (Conc)

$\phi_{vs} := 0.9$ Shear (Steel)

$\phi_m := 0.90$ Moment

Steel Reinforcement Properties

$f_y := 60000psi$

$Es := 29000ksi$

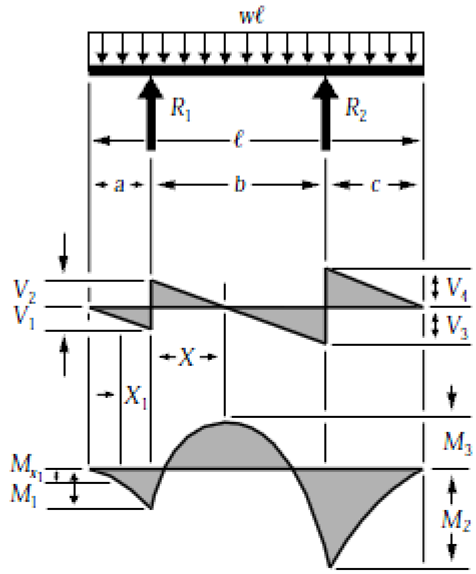
Surcharge Load

Load to simulate burial under 60 ft of fill

$q_L := 1.6 \cdot 60ft \cdot \gamma_{soil} = 11.52\,ksf$

$$LL_{rod} := \frac{q_L \cdot B_{soil} \cdot L_{soil}}{8} = 216 \cdot kip$$

## Steel Lid Calculations




### Assumptions

All calculations per foot of lid length

$w := q_L \cdot 1ft = 11.52 \cdot \frac{kip}{ft}$ $\quad \sigma y_{lid} := 50ksi$

### Lid Dimensions

$t_{lid} := 1in$ $\quad y_{lid} := \frac{t_{lid}}{2} = 0.5 \cdot in$

$b_{lid} := 12in$

$A_{lid} := t_{lid} \cdot b_{lid} = 12 \cdot in^2$

$a := 10in$ $\quad c := 10in$

$b := 20in$ $\quad L := a + b + c = 40 \cdot in$

$S_a := b$ anchor spacing

Conservative spacing since flanges of W-Beams will support the plate.

### Lid Shear Calculations

$R1_{lid} := w \cdot \frac{(a + b)}{2} = 14.4 \cdot kip$

$R2_{lid} := w \cdot b = 19.2 \cdot kip$

$R_{lid} := \max(R1_{lid}, R2_{lid}) = 19.2 \cdot kip$

$Vu_{lid} := w \cdot a = 9.6 \cdot kip$ $\quad Rlid := w \cdot \max\left[\frac{(a + b)}{2}, b\right] = 19.2 \cdot kip$

$Vn_{lid} := 0.6 \cdot \sigma y_{lid} \cdot A_{lid} = 360 \cdot kip$

$Vu_{lid} = 9.6 \cdot kip$

$\phi_{vs} \cdot Vn_{lid} = 324 \cdot kip$

Check = "Okay"

$Mulid_1 := \frac{-w \cdot a^2}{2} = -4 \cdot kip \cdot ft$ $\quad Mulid_2 := \frac{-w \cdot c^2}{2} = -4 \cdot kip \cdot ft$

$Mulid_3 := R_{lid} \cdot \left(\frac{R_{lid}}{2 \cdot w} - a\right) = 0 \cdot kip \cdot ft$

$Mu_{lid} := \max\left(\left|Mulid_1\right|, \left|Mulid_2\right|, \left|Mulid_3\right|\right) = 4 \cdot kip \cdot ft$

$I_{lid} := \frac{1}{12} b_{lid} \cdot t_{lid}^3 = 1 \cdot in^4$ $\quad My_{lid} := \frac{\sigma y_{lid} \cdot I_{lid}}{y_{lid}} = 8.333 \cdot kip \cdot ft$

$Mu_{lid} = 4 \cdot kip \cdot ft$

$\phi_m \cdot My_{lid} = 7.5 \cdot kip \cdot ft$

Check = "Okay"

## Reaction Beam Calculations

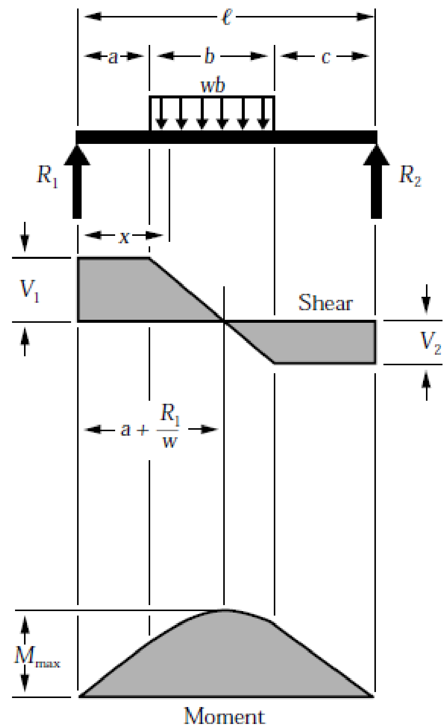
Assumptions

All calculations per foot of beam length

Additional strength from steel plates connecting channels is neglected in calculations

Beam is compact

Channels continuously braced by connecting plate




$$w_{beam} := \frac{R_{lid}}{1ft} = 19.2 \cdot \frac{kip}{ft}$$

$\sigma y_{beam} := 50ksi$

$Fy := 50ksi$

Beam Properties
From AISC Steel Manual 2005

Values taken from Table 3-2

W27x102 $\quad Lb_{beam} := 16.5ft \quad Ix := 3100in^4$

$\phi Mn_{beam} := 701kip \cdot ft$

$\phi Vn_{beam} := 419kip$

$a := .75ft \quad b := 15ft \quad c := .75ft$

Beam Shear Calculations

$$R1_{beam} := \frac{w_{beam} \cdot b}{2 \cdot Lb_{beam}}(2a + b) = 144 \cdot kip$$

$Vu_{beam} := R1_{beam} = 144 \cdot kip$

$\phi Vn_{beam} = 419 \cdot kip$

Check = "Okay"

Beam Moment Calculations

$$Mu_{beam} := R1_{beam} \cdot \left( a + \frac{R1_{beam}}{2 \cdot w_{beam}} \right) = 648 \cdot kip \cdot ft$$

$Mu_{beam} = 648 \cdot kip \cdot ft$

$\phi Mn_{beam} = 701 \cdot kip \cdot ft$

Check = "Okay"

Beam Deflection

$$\Delta_{beam} := \frac{5 \cdot w_{beam} \cdot Lb_{beam}^{4}}{384 \cdot Es \cdot Ix} = 0.356 \cdot in$$

## Anchor Beam Calculation

The anchor beam is to C Channels welded together so the post tensioning bar will be able to come through the middle of the beam. Since the bracing length of the beam is the spacing of the post tensioning bar, the beam will be braced every 20 inches.

$F_y := 50ksi$ $\quad Z := 23.3in^3$ $\quad t_w := 0.8in$ $\quad d_{ch} := 15in$ 2C15x33.9

$P_L := R1_{beam} = 144 \cdot kip$

$Vu_{AnBe} := \frac{P_L}{2} = 72 \cdot kip$

$Vn_{AnBe} := 0.6 \cdot F_y \cdot t_w \cdot d_{ch} = 360 \cdot kip$

$Vu_{AnBe} = 72 \cdot kip$

$\phi_{vs} \cdot Vn_{AnBe} = 324 \cdot kip$

Check = "Okay"

$Mu_{AnBe} := Vu_{AnBe} \cdot 10in = 60 \cdot kip \cdot ft$

$Mn_{AnBe} := F_y \cdot Z = 97.083 \cdot kip \cdot ft$

$Mu_{AnBe} = 60 \cdot kip \cdot ft$

$\phi_m \cdot Mn_{AnBe} = 87.375 \cdot kip \cdot ft$

Check = "Okay"

## Anchor Plate Calculation

The anchor plate is the plate that will span between the two channels to hold the anchor bolt.

Design as a 1 inch wide beam with a 3/4" span with the full 144 kip spread accross the length of the gap (conservative)

$L_{ap} := \frac{3}{4}in \quad B_{ap} := 1in \quad t_{ap} := 1.5in \quad \sigma_{ap} := 50ksi \quad w_{ap} := \frac{P_L}{L_{ap}} = 192 \cdot \frac{kip}{in}$

$Vu_{ap} := \frac{w \cdot L}{2} = 19.2 \cdot kip$

$\phi Vn_{ap} := \phi_{vs} \cdot 0.6 \cdot \sigma_{ap} \cdot B_{ap} \cdot t_{ap} = 40.5 \cdot kip$

$Vu_{ap} = 19.2 \cdot kip$

$\phi Vn_{ap} = 40.5 \cdot kip$

Check = "Okay"

$Mu_{ap} := \frac{w_{ap} \cdot L_{ap}^2}{8} = 13.5 \cdot kip \cdot in$

$\phi Mn_{ap} := 0.9 \cdot \sigma_{ap} \cdot \frac{B_{ap} \cdot t_{ap}^2}{6} = 16.875 \cdot kip \cdot in$

$Mu_{ap} = 13.5 \cdot kip \cdot in$

$\phi Mn_{ap} = 16.875 \cdot kip \cdot in$

Check = "Okay"

The plate will be a 3" Long x 3" Wide x 1.5' Thick

## Retaining Wall Calculations

Assumptions

All calculations per foot of wall length

Shear and moment calculations performed using ACI 318-08

Dimensions

$H_{wall} := 15ft$

$L_{wall} := 10ft$ $\quad b := 12in$

$B1_{wall} := 36in$ $\quad d := 41.4in$

$B2_{wall} := 24in$ $\quad d' := 45in$

Concrete Properties

$f_c = 6000\,psi$

$Ec = 4415201\,psi$

$\beta_1 = 0.75$

$\varepsilon_{cu} = 0.003$

Steel Properties

$f_y = 60000\,psi$

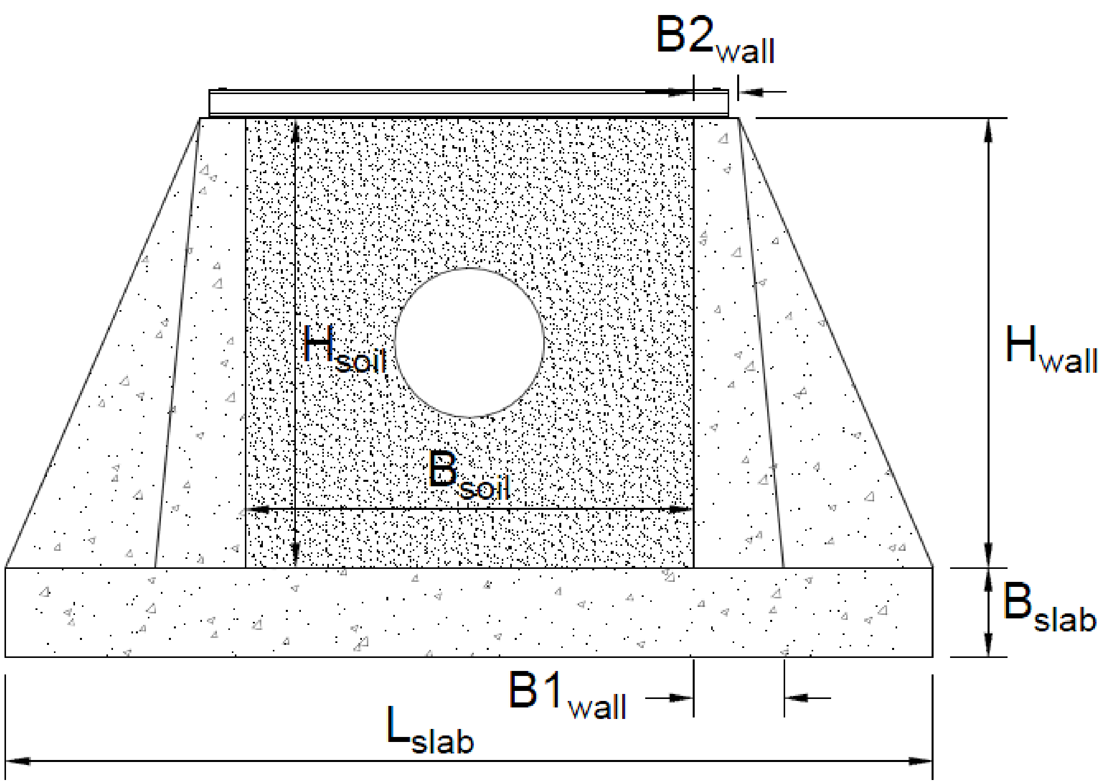
$Es = 29000 \cdot ksi$

$n := \dfrac{Es}{Ec}$

Strength Reduction Factors

$\phi_v := 0.75$

$\phi_m := 0.9$




Load Calculations

Since the wall is designed to deflect less than 1 mm, it will be designed as a at rest system.

$K_o := 0.5$ Clay Hard, Undrained

$q_t := 60ft \cdot \gamma_{soil} = 7200 \cdot psf$

Soil Bearing Pressure at 60 and 75 feet.

$q_b := 75ft \cdot \gamma_{soil} = 9000 \cdot psf$

Initial Soil Load

$$Vu_{wall} := 1.2 \cdot K_o \cdot \left[ q_t \cdot H_{wall} + \frac{(q_b - q_t) \cdot H_{wall}}{2} \right] = 72.9 \cdot \frac{kip}{ft}$$

$$Mu_{wall} := 1.2 \cdot K_o \cdot \left[ q_t \cdot \frac{H_{soil}^2}{2} + \frac{(q_b - q_t) \cdot H_{soil}^2}{2 \cdot 3} \right] = 526.5 \cdot \frac{kip \cdot ft}{ft}$$

$$Vu_{wall} = 73 \cdot \frac{kip}{ft}$$

$$Mu_{wall} = 526 \cdot \frac{kip \cdot ft}{ft}$$

Critical Shear and Moment Section at Base of Retaining Wall

Retaining Wall Shear Calculations

Overtuning

See if the mid section of the slab will handle any negative moment from the lateral forces on the wall.

$$M_o := H_{wall} \cdot K_o \cdot \left( q_t + \frac{q_b - q_t}{2} \right) \cdot \frac{H_{wall}}{3} = 303.75 \cdot \frac{kip \cdot ft}{ft}$$

$$M_R := q_b \cdot \frac{B_{soil}}{2} \cdot \left( \frac{L_{slab}}{2} - \frac{B_{soil}}{4} \right) = 793.125 \cdot \frac{kip \cdot ft}{ft}$$

The resisting moment just from the soil pressure is greater than the overturning moment.

$$Vn_{req} := \frac{Vu_{wall}}{\phi_v} = 97.2 \cdot \frac{kip}{ft} \qquad Vc_{wall} := 2\sqrt{\frac{f_c}{psi}} \cdot psi \cdot d = 77 \cdot \frac{kip}{ft}$$

$$Vs_{req} := Vn_{req} - Vc_{wall} = 20 \cdot \frac{kip}{ft}$$

$$4\sqrt{\frac{f_c}{psi}} \cdot psi \cdot b \cdot d = 154 \cdot kip$$

$4\sqrt{f_c} \cdot b \cdot d > Vs_{req}$ Level 3 Spacing

Use #5 Stirrups $As_v := 0.4in^2$ $f_{yt} := 60ksi$

$$s_{max} := \frac{d}{2} = 20.7 \cdot in \qquad s_{req} := \frac{As_v \cdot f_{yt} \cdot d}{Vs_{req} \cdot 1ft} = 49.1 \cdot in \qquad s_{prov} := 10in$$

$$Vs_{wall} := \frac{As_v \cdot f_{yt} \cdot d}{s_{prov} \cdot 1ft} = 99.36 \cdot \frac{kip}{ft}$$

$$Vn_{wall} := Vc_{wall} + Vs_{wall} = 176 \cdot \frac{kip}{ft}$$

$$Vu_{wall} = 73 \cdot \frac{kip}{ft}$$

$$\phi_v Vn_{wall} = 132 \cdot \frac{kip}{ft}$$

Check = "Okay"

#5 Stirrups @ 10 in. O.C.

Buttress and Wall Design

Vertical Rebar Design

$Slab_{edge} := 5ft$ Distance from edge of wall to edge of slab

$$h_{eff} := \cos\left(\operatorname{atan}\left(\frac{H_{wall}}{Slab_{edge}}\right)\right)\cdot H_{wall} + B2_{wall} = 6.743\,ft$$

$Sp_{bwall} := 2.75ft$ Center to center spacing of butress walls

$w_1 := q_t\cdot Sp_{bwall} = 19.8\cdot\frac{kip}{ft}$ Distributed Rectangular Load

$w_2 := (q_b - q_t)\cdot Sp_{bwall} = 4.95\cdot\frac{kip}{ft}$ Distributed Trainagle Load

$$M_1 := \frac{w_1\cdot H_{wall}^2}{2} = 2227.5\cdot kip\cdot ft$$

$$M_2 := \frac{w_2\cdot H_{wall}^2}{6} = 185.625\cdot kip\cdot ft$$

$$Mu_{bt} := 1.2\cdot(M_1 + M_2) = 2896\cdot kip\cdot ft$$

$b_{bt} := 12in$ $d_{eff} := h_{eff} - 3in$ $d'_{eff} := 3in$

$A_s := 9in^2$ $A'_s := 4in^2$

The buttress is being analyzed as a T cross-section at the bottom taking the max load which is conservative since the wall and buttress will be working together making a T-Beam design more accurate. Also an effective height was found which is also more conservative. The compression

Guess $c := 7.3in$

$$\varepsilon_s := \varepsilon_{cu}\cdot\left(\frac{d_{eff} - c}{c}\right) = 0.02902 > \quad \varepsilon_y := \frac{f_y}{Es} = 0.00207$$

$$\left|\begin{array}{ll}\text{"Tensions Steel Yielded"} & \text{if } \varepsilon_s \ge \varepsilon_y \\ \text{"Not Yielded"} & \text{otherwise}\end{array}\right. = \text{"Tensions Steel Yielded"}$$

$$f_s := Es\cdot\varepsilon_s = 841.648\cdot ksi$$

$$T := \min(A_s\cdot f_s, f_y\cdot A_s) = 540\cdot kip$$

$$\varepsilon'_s := \varepsilon_{cu}\cdot\left(\frac{c - d'_{eff}}{c}\right) = 0.00177$$

$$\left|\begin{array}{ll}\text{"Compression Steel Yielded"} & \text{if } \varepsilon'_s \ge \varepsilon_y \\ \text{"Not Yielded"} & \text{otherwise}\end{array}\right. = \text{"Not Yielded"}$$

$f'_s := Es \cdot \varepsilon'_s = 51.247 \cdot ksi$

$C_s := \min\left(f'_s \cdot A'_s, f_y \cdot A'_s\right) = 204.986 \cdot kip$

$C_c := 0.85 \cdot f_c \cdot \beta_1 \cdot c \cdot b_{bt} = 335.07 \cdot kip$

The equation below must be close to zero within +-100 pounds.

$T - C_s - C_c = -56.301\, lbf$

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b_{bt}} = 8.824 \cdot in$$

$$\phi Mn_{bt} := \phi_m \left[ C_s \cdot \left(d_{eff} - d'_{eff}\right) + C_c \cdot \left(d_{eff} - \frac{a}{2}\right)\right] = 2999 \cdot kip \cdot ft$$

$$\phi_n := \frac{0.003}{c} = 0 \cdot \frac{1}{in}$$

$Mu_{bt} = 2896 \cdot kip \cdot ft$

$\phi Mn_{bt} = 2999 \cdot kip \cdot ft$

Check = "Okay"

Effective Flange Width

$b_{f1} := 16 \cdot B2_{wall} + b_{bt} = 33\, ft$

$b_{f2} := Sp_{bwall} = 2.75\, ft$

$$b_{f3} := \frac{H_{wall}}{4} = 3.75\, ft$$

$b_f := \min\left(b_{f1}, b_{f2}, b_{f3}\right) = 2.75\, ft$

#9 vertical bars spaced at 3.5 inches on center on soil side of wall

4 #9 vertical bars spaced at 1 inch center to center and rebar line spaced at 12 inches in the buttress

This is a significant more steel in the buttress than what was analyzed (for simplification) giving the buttress a significantly more strength than calculated.

Temperature and Shrinkage Steel

$0.0018 \cdot 1ft \cdot B1_{wall} = 0.778 \cdot in^2$

There is adequate amount of steel in the wall.

Since the slab is thick steel will be placed on the opposite end of the soil.

#6 vertical bars spaced at 12 inches (opposite side of soil)

Shear Strength of Buttress Wall

$$Vu_{bt} := 1.2 \cdot K_o \cdot \left[ q_t \cdot H_{wall} + \frac{(q_b - q_t) \cdot H_{wall}}{2} \right] \cdot Sp_{bwall} = 200.475 \cdot kip$$

$$Vc_{bt} := 2 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot b_{bt} \cdot d_{eff} = 144.858 \cdot kip$$

$$Vn_{req} := \frac{Vu_{bt}}{\phi_v} = 267.3\, ft \cdot \frac{kip}{ft}$$

$$Vs_{req} := Vn_{req} - Vc_{bt} = 122\, ft \cdot \frac{kip}{ft}$$

Use #6 Bars $As_v := 0.88 in^2$ $f_{yt} := 60ksi$

$$s_{max} := \frac{d_{eff}}{2} = 38.96 \cdot in \qquad s_{req} := \frac{As_v \cdot f_{yt} \cdot d_{eff}}{Vs_{req}} = 33.6 \cdot in \qquad s_{prov} := 12in$$

$$Vs_{bt} := \frac{As_v \cdot f_{yt} \cdot d_{eff}}{s_{prov}} = 343 \cdot kip$$

$$Vn_{bt} := Vc_{bt} + Vs_{bt} = 488 \cdot kip$$

$Vu_{bt} = 200 \cdot kip$

$\phi_v\, Vn_{bt} = 366 \cdot kip$

Check = "Okay"

#6 Bars @ 12 in. O.C.

Deflection

The extra foot of concrete is being added to get a more accurate answer for the deflection

$$h_{eff} := \cos\left(\operatorname{atan}\left(\frac{H_{wall}}{Slab_{edge}}\right)\right) \cdot H_{wall} + B1_{wall} = 7.743\, ft$$

$$c_g := \frac{b_f \cdot B2_{wall} \cdot \frac{B2_{wall}}{2} + b_{bt} \cdot h_{eff} \cdot \left(B2_{wall} + \frac{h_{eff}}{2}\right)}{b_f \cdot B2_{wall} + b_{bt} \cdot h_{eff}} = 3.85\, ft$$

$$I_g := \frac{b_f \cdot B2_{wall}^3}{12} + b_f \cdot B2_{wall} \cdot \left(c_g - \frac{B2_{wall}}{2}\right)^2 + \frac{b_{bt} \cdot h_{eff}^3}{12} + b_{bt} \cdot h_{eff} \cdot \left[c_g - \left(B2_{wall} + \frac{h_{eff}}{2}\right)\right]^2$$

$$I_g = 2422967 \cdot in^4$$

$$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi = 581\, psi$$

$$M_{cr} := \frac{f_r \cdot I_g}{c_g} = 2540 \cdot kip \cdot ft$$

Yield Calculations

$A_s = 9 \cdot in^2 \qquad A'_s = 4 \cdot in^2 \qquad d_{eff} = 78 \cdot in \qquad d'_{eff} = 3 \cdot in$

Guess $c_{cr} := 3in$ so the equation below equals 0

$$\frac{b_f \cdot c_{cr} \cdot \left(\frac{c_{cr}}{2}\right) + n \cdot A_s \cdot d_{eff} + n \cdot A'_s \cdot d'_{eff}}{b_f \cdot c_g + n \cdot A_s + n \cdot A'_s} = 3.003 \cdot in$$

$c_{cr} = 3 \cdot in$ distance to centroid of cracked section

Cracked Moment of Inertia

$$I_{cr} := \frac{b_f \cdot c_{cr}^3}{3} + n \cdot A_s \cdot \left(d_{eff} - c_{cr}\right)^2 + n \cdot A'_s \cdot \left(d'_{eff} - c_{cr}\right)^2 = 3.32 \times 10^5 \cdot in^4$$

Effective Moment of Inertia

$$I_e := \min\left[\left(\frac{M_{cr}}{Mu_{bt}}\right)^3 \cdot I_g + \left[1 - \left(\frac{M_{cr}}{Mu_{bt}}\right)^3\right] \cdot I_{cr}, I_g\right] = 1743146 \cdot in^4$$

$$\Delta := \frac{Mu_{bt} \cdot H_{wall}^2}{1.2Ec \cdot I_e} = 0.122 \cdot in$$

This deflection is more than wanted but this is a very conservative number since there is a significant amount of steel withing the wall and buttress that was not added to the analysis.

Horizontal Rebar Design

The wall will need horizontal rebar to transfer the load to the buttresses. The sections of the wall between the butresses will be analyzed as a fixed-fixed beam with one foot width at the bottom where the maximum load is.

$w_L := q_b \cdot 1ft = 9 \cdot \frac{kip}{ft} \qquad Sp_{bwall} = 2.75\,ft$

$$\phi M_{neg} := \phi_m \cdot \frac{w_L \cdot Sp_{bwall}^2}{12} = 5.1 \cdot kip \cdot ft$$

$$\phi M_{pos} := \phi_m \cdot \frac{w_L \cdot Sp_{bwall}^2}{24} = 2.6 \cdot kip \cdot ft$$

$b_{ff} := 12in \qquad h_{ff} := 2ft \qquad d_{ff} := h_{ff} - 3in \qquad d'_{eff} := 3in$

$A_s := 0.88in^2 \qquad A'_s := 0.88in^2$ #6 bars @ 6" C-C

Guess $c := 1.993in$

$$\varepsilon_s := \varepsilon_{cu} \cdot \left( \frac{d_{eff} - c}{c} \right) = 0.11429 > \quad \varepsilon_y := \frac{f_y}{Es} = 0.00207$$

$$\left| \begin{array}{l} \text{"Tensions Steel Yielded"} \quad \text{if } \varepsilon_s \ge \varepsilon_y \\ \text{"Not Yielded"} \quad \text{otherwise} \end{array} \right. = \text{"Tensions Steel Yielded"}$$

$$f_s := Es \cdot \varepsilon_s = 3314.469 \cdot ksi$$

$$T := \min\left(A_s \cdot f_s, f_y \cdot A_s\right) = 52.8 \cdot kip$$

$$\varepsilon'_s := \varepsilon_{cu} \cdot \left( \frac{c - d'_{eff}}{c} \right) = -0.00152$$

$$\left| \begin{array}{l} \text{"Compression Steel Yielded"} \quad \text{if } \varepsilon'_s \ge \varepsilon_y \\ \text{"Not Yielded"} \quad \text{otherwise} \end{array} \right. = \text{"Not Yielded"}$$

$$f'_s := Es \cdot \varepsilon'_s = -43.958 \cdot ksi$$

$$C_s := \min\left(f'_s \cdot A'_s, f_y \cdot A'_s\right) = -38.683 \cdot kip$$

$$C_c := 0.85 \cdot f_c \cdot \beta_1 \cdot c \cdot b_{bt} = 91.479 \cdot kip$$

The equation below must be close to zero within +-100 pounds.

$$T - C_s - C_c = 4.652\ lbf$$

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b_{bt}} = 0.863 \cdot in$$

$$M_n := C_s \cdot \left(d_{eff} - d'_{eff}\right) + C_c \cdot \left(d_{eff} - \frac{a}{2}\right) = 349 \cdot kip \cdot ft$$

## **Development Length Calculation**

Development Length of Retaining Wall Dowels into the Footing

#9 bars $f_y = 60000\,psi$ $\Psi_e := 1$ $\Psi_c := 0.7$ $\Psi_r := 1$ $d_b := 1.128in$

$$L_{dh} := \left( \frac{f_y \cdot \Psi_e \cdot \Psi_c \cdot \Psi_r}{50 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi} \right) \cdot d_b = 12.2 \cdot in$$

Hooked Bars

$$L_{ext} := 12 \cdot d_b = 13.5 \cdot in$$

$\Psi_t := 1$ $\Psi_e := 1$ $\Psi_s := 1$ $c_b := 3in$ $K_{tr} := 0$

$$L_d := \left[ \frac{3}{40} \cdot \frac{f_y}{\sqrt{\frac{f_c}{psi}} \cdot psi} \cdot \frac{\Psi_t \cdot \Psi_e \cdot \Psi_s}{\left( \frac{c_b + K_{tr}}{d_b} \right)} \cdot d_b \right] = 24.6 \cdot in$$

Straight Bar

#6 bars $d_b := 0.75in$

$$L_{dh} := \left( \frac{f_y \cdot \Psi_e \cdot \Psi_c \cdot \Psi_r}{50 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi} \right) d_b = 8.1 \cdot in$$

Hooked Bars

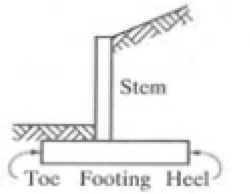
$$L_{ext} := 12 \cdot d_b = 9 \cdot in$$

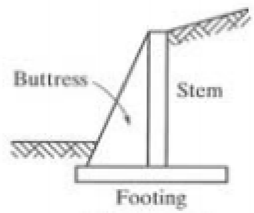
$\Psi_t := 1$ $\Psi_e := 1$ $\Psi_s := 0.8$ $c_b := 3in$ $K_{tr} := 0$

$$L_d := \left[ \frac{3}{40} \cdot \frac{f_y}{\sqrt{\frac{f_c}{psi}} \cdot psi} \cdot \frac{\Psi_t \cdot \Psi_e \cdot \Psi_s}{\left( \frac{c_b + K_{tr}}{d_b} \right)} \cdot d_b \right] = 8.7 \cdot in$$

Straight Bar

## Slab Calculations




Slab Toe Calculations

$$d := B_{slab} - 3in = 45 \cdot in$$

$w_{conc} := 150pcf$ Weight of Concrete

$$W_1 := B_{slab} \cdot \frac{L_{slab}}{2} \cdot w_{conc} = 9.3 \cdot \frac{kip}{ft}$$

$$W_2 := \frac{1}{2} \cdot \left(B1_{wall} - B2_{wall}\right) \cdot H_{wall} \cdot w_{conc} = 1.125 \cdot \frac{kip}{ft}$$

$$W_3 := B2_{wall} \cdot H_{wall} \cdot w_{conc} = 4.5 \cdot \frac{kip}{ft}$$

$$W_4 := q_b \cdot \frac{B_{soil}}{2} = 67.5 \cdot \frac{kip}{ft}$$

$$R_v := \sum W = 82.425 \cdot \frac{kip}{ft}$$

Moment Arm

$$MA_1 := \frac{L_{slab}}{4} = 7.75\,ft \qquad MA_2 := \frac{L_{slab}}{2} - \frac{B_{soil}}{2} - \frac{3}{4} \cdot B1_{wall} = 5.75\,ft$$

$$MA_3 := \frac{L_{slab}}{2} - \frac{B_{soil}}{2} - \frac{B2_{wall}}{2} = 7\,ft \qquad MA_4 := \frac{L_{slab}}{2} - \frac{B_{soil}}{2} = 8\,ft$$

$MR := \sum_{n=1}^{4} \left(W_n \cdot MA_n\right) = 650.044 \cdot \frac{kip \cdot ft}{ft}$ Righting Moment

Footing Soil Pressures

$$x_{bar} := \frac{MR - M_o}{R_v} = 4.201\,ft$$

$$SP_{max} := \frac{-R_v}{1ft \cdot \frac{L_{slab}}{2}} - \frac{R_v \cdot \left(\frac{L_{slab}}{2} - x_{bar}\right) \cdot \frac{L_{slab}}{2}}{\left[\frac{1ft \cdot \left(\frac{L_{slab}}{2}\right)^3}{12}\right]} = -51.834 \cdot \frac{ksf}{ft}$$

$$SP_{min} := \frac{-R_v}{1ft\cdot\frac{L_{slab}}{2}} + \frac{R_v\cdot\left(\frac{L_{slab}}{2} - x_{bar}\right)\cdot\frac{L_{slab}}{2}}{\left[\frac{1ft\cdot\left(\frac{L_{slab}}{2}\right)^3}{12}\right]} = 41.198\cdot\frac{ksf}{ft}$$

Slab Toe Shear Calculations

$$Slope := \frac{SP_{min} - SP_{max}}{\frac{L_{slab}}{2}} = 0.5\cdot\frac{ksf}{ft\cdot in}$$

$$Vu_{toe} := \left[\left(SP_{max} + Slope\cdot 4ft\right)\cdot 4ft + \frac{1}{2}\cdot Slope\cdot 4ft\cdot 4ft\right]1ft = -63.286\cdot\frac{kip}{ft}$$

$$Vc_{toe} := 2\cdot\sqrt{\frac{f_c}{psi}}\cdot psi\cdot d = 83.7\cdot\frac{kip}{ft}$$

Use #4 Stirrups $As_v := 0.4in^2$ $f_{yt} := 60ksi$

$$s_{max} := \frac{d}{2} = 22.5\cdot in \qquad s_{prov} := 10in$$

$$Vs_{toe} := \frac{As_v\cdot f_{yt}\cdot d}{s_{prov}\cdot 1ft} = 108\cdot\frac{kip}{ft}$$

$$Vn_{toe} := Vc_{toe} + Vs_{toe}$$

$$Vu_{toe} = -63\cdot\frac{kip}{ft}$$

$$\phi_v\cdot Vn_{toe} = 144\cdot\frac{kip}{ft}$$

Check = "Okay"

Slab Toe Moment Calculations

$$Mu_{toe} := \left[\left(SP_{max} + Slope\cdot 4ft\right)\cdot 4ft\cdot\frac{4ft}{2} + \frac{1}{2}\cdot Slope\cdot 4ft\cdot 4ft\cdot\frac{4ft}{2}\right]1ft = -126.571\cdot\frac{kip\cdot ft}{ft}$$

$As_{toe} := 2\,0.44in^2$ 2 #6 bars per foot

$$a := \frac{As_{toe}\cdot f_y}{0.85\cdot f_c\cdot b} = 0.86\cdot in \qquad c := \frac{a}{\beta_1} = 1.15\cdot in$$

$$\varepsilon s := \varepsilon_{cu}\cdot\left(\frac{d - c}{c}\right) = 0.11436\cdot\frac{in}{in}$$

$$Mn_{toe} := \frac{As_{toe}\cdot f_y\cdot\left(d - \frac{a}{2}\right)}{b} = 196.1\cdot\frac{kip\cdot ft}{ft}$$

$$Mu_{toe} = -127 \cdot \frac{kip \cdot ft}{ft}$$

$$\phi_m \cdot Mn_{toe} = 176 \cdot \frac{kip \cdot ft}{ft}$$

Check = "Okay"

Slab Heel Shear Calculations

$$Vu_{heel} := 1.2\left(q_b \cdot \frac{B_{soil}}{2} + w_{conc} \cdot B_{slab} \cdot \frac{B_{soil}}{2}\right) = 86.4 \cdot \frac{kip}{ft}$$

$$Vc_{heel} := 2 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot d = 83.7 \cdot \frac{kip}{ft}$$

Use #4 Stirrups $As_v := 0.4in^2$ $f_{yt} := 60ksi$

$$s_{max} := \frac{d}{2} = 22.5 \cdot in \qquad s_{prov} := 10in$$

$$Vs_{heel} := \frac{As_v \cdot f_{yt} \cdot d}{s_{prov} \cdot 1ft} = 108 \cdot \frac{kip}{ft}$$

$$Vn_{heel} := Vc_{heel} + Vs_{heel}$$

$$Vu_{heel} = 86 \cdot \frac{kip}{ft}$$

$$\phi_v \cdot Vn_{heel} = 144 \cdot \frac{kip}{ft}$$

Check = "Okay"

Slab Heel Moment Calculations

Two parrallel layers are needed. $d := d - 0.5in$

$$Mu_{heel} := Vu_{heel} \cdot \frac{B_{soil}}{2} = 648 \cdot \frac{kip \cdot ft}{ft}$$

There is additional moment found from the heel calculation since the lateral force was so intense

$$Mu_{heel2} := \frac{1}{2} SP_{min} \cdot \left(\frac{B_{soil}}{2}\right) \cdot \frac{B_{soil}}{3} \cdot 1ft = 772 \cdot \frac{kip \cdot ft}{ft}$$

$$Mu_{heel} := \max\left(Mu_{heel}, Mu_{heel} + Mu_{heel2}\right) = 1420 \cdot \frac{kip \cdot ft}{ft}$$

$As_{heel} := 8 \cdot 1in^2$ 8 #9 bars per foot (2 parrallel layers of 4 #9 bars per foot)

$$a := \frac{As_{heel} \cdot f_y}{0.85 \cdot f_c \cdot b} = 7.84 \cdot in \qquad c := \frac{a}{\beta_1} = 10.46 \cdot in$$

$$\varepsilon s := \varepsilon_{cu} \cdot \left(\frac{d - c}{c}\right) = 0.00977 \cdot \frac{in}{in}$$

$$Mn_{heel} := \frac{As_{heel} \cdot f_y \cdot \left(d - \frac{a}{2}\right)}{b} = 1623.14 \cdot \frac{kip \cdot ft}{ft}$$

## Post Tensioning Rebar Calculation

The bars anchoring the steel beams down go are post-tensioning bars that go from the top all the way to the bottom of the slab. The bottom part of the post-tensioning bars will be embedded in the slab so the slab will still have a flat bottom.

There will be three tendons per anchorage loaction.

$f_{uta} := 270\text{ksi}$  $dia_a := 0.6\text{in}$  $A_a := \frac{\pi}{4} \cdot dia_a^2 = 0.283 \cdot \text{in}^2$  $n_a := 4$  # of anchors

$P_L = 144 \cdot \text{kip}$  Total load per anchorage location  $S_a = 20 \cdot \text{in}$  anchor spacing

$\phi_m \cdot n_a \cdot f_{uta} \cdot A_a = 274.827 \cdot \text{kip}$

Assume the jacking force will only be 60% of the ultimate strength in post-tensioning

$f_j := 0.6 f_{uta} \cdot A_a \cdot n_a = 183.218 \cdot \text{kip}$

The jacking force needs to be greater than the load cause the compression on the concrete from the jacking force needs to be more than the tension on the concrete due to the load.

Assuming the strain in the concrete and steel evenly change cause they elongate the same.

Stress in the concrete (- means compression)

$$A_{tr} := S_a \cdot B2_{wall} + n \cdot A_a = 481.857 \cdot \text{in}^2$$

$$\sigma_{cj} := \frac{-f_j}{A_{tr}} = -380\,\text{psi}$$

$$\Delta\sigma_c := \frac{P_L}{S_a \cdot B2_{wall}} = 300\,\text{psi}$$

$$\sigma_c := \sigma_{cj} + \Delta\sigma_c = -80\,\text{psi}$$

Stress in the concrete (- means compression)

$$\sigma_{asj} := \frac{f_j}{A_a \cdot n_a} = 162 \cdot \text{ksi}$$

$$\Delta\sigma_{as} := \frac{n \cdot P_L}{A_{tr}} = 1.963 \cdot \text{ksi}$$

$$\sigma_{as} := \sigma_{asj} + \Delta\sigma_{as} = 164 \cdot \text{ksi}$$

Even thought the bar will have more tension added after post-tensioning to the load the additional stress in the bar is negligible and the concrete stays in compression.

## Anchorage of Post Tensioning Bar (ACI 318-14) Calculation

The bare will be anchored at the bottom but will be withing the slab ao assume anchorage starts one foot above the ground.

$N_{ua} := P_L = 144 \cdot kip$

The design will be as if the anchorage is anchoring the wall down so the free body diagram will be cut at the wall-slab joint.

$h_{ef} := \min\left(B_{slab} - 1ft, \frac{2}{3} \cdot B_{slab}, B_{slab} - 4in\right) = 32 \cdot in$ (17.7.5)

$Sp_{min} := 4 \cdot dia_a = 2.4 \cdot in$ Minimum spacing (17.7.1)

$Sp_a := 2.5in$

$Edge\_Distance\_Req := 3in$ (17.7.2)

$c_{a.min} := 2ft$

Tension Strength of Anchor (17.4.1)

$N_{sa} := A_a \cdot f_{uta} = 76 \cdot kip$

$\phi_t := 0.75$ (17.3.2.3.a.i)

$\phi N_{sa} := \phi_t \cdot N_{sa} = 57 \cdot kip$

$\phi N_{sag} := \phi N_{sa} \cdot n_a = 229 \cdot kip$

$\begin{cases} \text{"okay"} & \text{if } \phi N_{sa} \cdot n \geq N_{ua} \\ \text{"not okay"} & \text{otherwise} \end{cases} = \text{"okay"}$

$k_c := 24$ (17.4.2.2)

$N_b := k_c \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot in^2 = 337 \cdot kip$ (17.4.2.2)

Assume bearing plate is 3*spacing of the anchor fot the length and negligible for the width in calculating Anc

$A_{nc} := (1.5 \cdot h_{ef} + 3 \cdot Sp_a + 2ft) \cdot (2 \cdot 1.5 \cdot h_{ef}) = 7632 \cdot in^2$ (17.4.2.1)

$A_{nco} := 9 \cdot h_{ef}^{\ 2} = 9216 \cdot in^2$ (17.4.2.1)

$\psi_{ecN} := 1$ (17.4.2.4)

$\psi_{edN} := \begin{cases} 1 & \text{if } c_{a.min} \geq 1.5 \cdot h_{ef} \\ \left(0.7 + 0.3 \cdot \frac{c_{a.min}}{1.5 \cdot h_{ef}}\right) & \text{otherwise} \end{cases} = 0.85$ (17.4.2.5)

$\psi_{cN} := 1$ (conservative) (17.4.2.6)

$\psi_{cpN} := 1$ cast-in place anchors (17.4.2.7)

$$N_{cb} := \frac{A_{nc}}{A_{nco}} \cdot \psi_{edN} \cdot \psi_{cN} \cdot \psi_{cpN} \cdot N_b = 237 \cdot kip \quad (17.4.2.1)$$

$$\phi N_{cb} := \phi_t \cdot N_{cb} = 178 \cdot kip$$

Pullout Strength of Anchor (17.4.3)

Since there will be a bearing plate connectiong the tendons the pullout strength will be analyzed as a group.

$\Psi_{c.P} := 1$ conservative (17.4.3.6)

$A_{brg} := 30in^2$ Estimate from VSL Bonded Slab Post Tensioning Catalog

$$N_p := 8 \cdot A_{brg} \cdot f_c = 1440 \cdot kip \quad (17.4.3.4)$$

$$\phi_p := 0.7 \quad (17.3.3.c.ii)$$

$$\phi N_{pn} := \phi_p \cdot \Psi_{c.P} \cdot N_p = 1008 \cdot kip$$

Concrete Side-Face Blowout (17.4.4)

$1.5 \cdot 2ft = 3\,ft$ > $h_{ef} = 2.667\,ft$

So concrete side-face blowout strength is adequate

Tensile Strength Check

$\phi N_n := \min(\phi N_{sag}, \phi N_{cb}, \phi N_{pn}) = 178 \cdot kip$ > $N_{ua} = 144 \cdot kip$

$$\begin{array}{|ll} \text{"OK"} & \text{if } \phi N_n > N_{ua} \\ \text{"NG"} & \text{otherwise} \end{array} = \text{"OK"}$$

# Appendix G: Test Frame Construction Cost Analysis

***CostWorks 2015 - Auburn***

| Qty | CSI Number | Description | Unit | Bare Mat. | Bare Labor | Bare Equip. | Total | Total Incl. O&P | Type | Release |
|---|---|---|---|---|---|---|---|---|---|---|
| ***Concrete*** | | | | | | | | | | |
| 175.000 | 03311 335 0411 | Structural concrete, ready mix, heavyweight, 6000 PSI, includes local aggregate, sand, Portland cement (Type I) and water, delivered, excludes all additives and treatments | C.Y. | 20,300.00 | 0.00 | 0.00 | 20,300.00 | 22,400.00 | Open | 2016 |
| 65.000 | 03311 370 4600 | Structural concrete, placing, slab on grade, direct chute, over 6" thick, includes leveling (strike off) & consolidation, excludes material | C.Y. | 0.00 | 572.00 | 24.70 | 596.70 | 981.50 | Open | 2016 |
| 110.000 | 03311 370 5300 | Structural concrete, placing, walls, direct chute, 15" thick, includes leveling (strike off) & consolidation, excludes material | C.Y. | 0.00 | 1,523.50 | 64.90 | 1,588.40 | 2,585.00 | Open | 2016 |
| 360.000 | 03111 345 0020 | C.I.P. concrete forms, footing, continuous wall, plywood, 1 use, includes erecting, bracing, stripping and cleaning | SFCA | 2,448.00 | 997.20 | 0.00 | 3,445.20 | 4,374.00 | Open | 2016 |
| 150.000 | 03111 385 2400 | C.I.P. concrete forms, wall, job built, plywood, over 8' to 16' high, 1 use, includes erecting, bracing, stripping and cleaning | SFCA | 454.50 | 840.00 | 0.00 | 1,294.50 | 1,912.50 | Open | 2016 |
| 3.500 | 03211 160 0600 | Reinforcing steel, in place, #6, A615, grade 60, incl labor for accessories, excl material for accessories | Ton | 3,395.00 | 1,925.00 | 0.00 | 5,320.00 | 7,000.00 | Open | 2016 |
| 4.600 | 03211 160 0750 | Reinforcing steel, in place, #9, A615, grade 60, incl labor for accessories, excl material for accessories | Ton | 4,462.00 | 1,449.00 | 0.00 | 5,911.00 | 7,360.00 | Open | 2016 |
| ***Pipe*** | | | | | | | | | | |
| 15.000 | 33411 360 2090 | Public storm utility drainage piping, reinforced concrete pipe (RCP), 60" diameter, 8' lengths, class 3, excludes excavation or backfill, gaskets | L.F. | 2,040.00 | 555.00 | 352.50 | 2,947.50 | 3,570.00 | Open | 2016 |
| 15.000 | 22111 378 0182 | Pipe, plastic, high density polyethylene (HDPE), single wall, straight, welded, based on 40' length, 60" diam., DR 26, add 1 weld per joint, excludes hangers, trenching, backfill, hoisting or digging equipment | L.F. | 1,785.00 | 255.00 | 150.00 | 2,190.00 | 2,565.00 | Open | 2016 |
| ***Soils*** | | | | | | | | | | |
| 75.000 | G1030 815 2920 | Pipe bedding, 15' wide, pipe size 60" diameter, using backhoe | C.Y. | 4,725.00 | 2,362.50 | 0.000 | 7,087.50 | 7,796.250 | Open | 2016 |
| ***Steel*** | | | | | | | | | | |
| 120.000 | 05122 375 4760 | Structural steel beam or girder, W21x101, A992 steel, shop fabricated, incl shop primer, bolted connections | L.F. | 17,640.00 | 343.20 | 199.20 | 18,182.40 | 20,280.00 | Open | 2016 |
| 30.000 | 05122 340 0474 | Angle framing, structural steel, 3"x2"x1/4", field fabricated, incl cutting & welding | L.F. | 140.70 | 445.50 | 67.20 | 653.40 | 1,065.00 | Open | 2016 |
| 225.000 | 05122 365 0500 | Steel plate, structural, 1.5" T, shop fabricated, incl shop primer | S.F. | 18,225.00 | 0.00 | 0.00 | 18,225.00 | 20,081.25 | Open | 2016 |
| 48.000 | 05122 340 0672 | Channel framing, structural steel, field fabricated, C15x33.9, incl cutting & welding | L.F. | 439.20 | 1,296.00 | 194.40 | 1,929.60 | 3,120.00 | Open | 2016 |
| 400.000 | 03230 550 0100 | Prestressing steel, grouted strand, 300 kip, post-tensioned in field | Lb. | 3,168.00 | 2,268.00 | 108.00 | 5,544.00 | 7,440.00 | Open | 2016 |
| | | **Totals** | | **$79,222.40** | **$14,831.90** | **$1,160.90** | **$95,215.20** | **$112,530.50** | | |
| | | **Total Construction** | | **$70,672.40** | **$11,659.40** | **$658.40** | **$82,990.20** | **$98,599.25** | | |
| | | **Total Pipe/Soil Installation (per installation)** | | **$6,510.00** | **$2,617.50** | **$150.00** | **$9,277.50** | **$10,361.25** | | |

| Concrete | | | | | |
|---|---|---|---|---|---|
| Item | Volume (ft^3) | Quantity | Total (ft^3) | CY | |
| Slab | 1736 | 1 | 1736 | 64.30 | |
| Retaining wall | 487.5 | 6 | 2925 | 108.33 | |
| Wall | 22.5 | 2 | 45 | 1.67 | |
| | Total Volume (ft^3) | | 4706 | 174.30 | |

| Rebar | | | | | | |
|---|---|---|---|---|---|---|
| Item | Length (ft) | Quantity | Total (ft) | | Fill Volume | |
| Ancor Rods | 21 | 40 | 840 | | 1955.48 | |
| #6 Bars@6" (Wall) | 10 | 116 | 1160 | | 72.43 | |
| #6Bars@12" (Wall) | 19.25 | 18 | 346.5 | | | |
| 2#6@12" (Wall) | 123.25 | 12 | 1479 | | | |
| 4#9 Bars (Wall) | 19.71 | 24 | 473.04 | | | |
| 4#9 Bars@12" (Wall) (Per Butress Wall) | 23.08 | 6 | 138.48 | | | |
| #9 Bars@3.5" (Wall) | 19.63 | 66 | 1295.58 | | | |
| #9 Bars@6" (Base) | 31 | 26 | 806 | | | |
| #6 Bars@6" (Base) | 31 | 26 | 806 | | | |
| #6Bars@12" (Base) | 14 | 60 | 840 | | | |
| | | Total #6 | 4631.5 | 1.5 | 6947.25 | lbs |
| | | Total #9 | 2713.1 | 3.4 | 9224.54 | lbs |

| Steel | | | |
|---|---|---|---|
| Item | Length (ft) | Quantity | Total (ft) |
| Reaction Beam, W21x102 | 19.5 | 6 | 117 |
| Anchor Beam, 2C15x33.9 | 12 | 2 | 24 |
| L3x2x1/4 angle | 15 | 2 | 30 |
| | | | |
| 1.5" steel plate A992 | 15'x15' | 225 | SF |
| 6 fiber tendon rods | 4 per | 10 | 40 |