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STP 1528

Plastic Pipe and Fittings

Past, Present, and Future

JAI Guest Editor

Thomas S. Walsh

**Journal of ASTM International
Selected Technical Papers STP1528
Plastic Pipe and Fittings: Past, Present,
and Future**

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Foreword

THIS COMPILATION OF THE *JOURNAL OF ASTM INTERNATIONAL (JAI)*, STP1528, *Plastic Pipe and Fittings: Past, Present, and Future* contains only the papers published in JAI that were presented at a Symposium on Plastic Pipe and Fittings: Yesterday, Today, and Tomorrow held during November 9, 2009 in Atlanta, GA, USA. The Symposium was sponsored by ASTM International Committee F17 on Plastic Piping Systems.

Thomas S. Walsh, Walsh Consulting Services, Houston, TX served as the Symposium Chairman and JAI Guest Editor.

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Overview

This symposium is a continuation of two previous ASTM symposia, “Buried Plastic Pipe Technology,” (STP1093) held in 1990 and “Buried Plastic Pipe Technology, 2nd Volume,” (STP1222) held in 1994. The first symposium on Buried Plastic Technology was organized to provide the users of water, sewer, drainage waste management, irrigation and gas projects with the current state of the art engineering data and techniques for the use of plastic piping materials. The second symposia followed as a sequel to the first repeating the same intent. The current symposia has similar intentions but also was organized to capture historical viewpoints on the last sixty plus years of the successful introduction and use of plastic piping products in North America. The papers are organized into five sections: Historical Reviews, Design, Testing and Failure Analysis, New Materials and Applications, and Installation.

In the Historical Review section, Bob Walker presented an overview of the successful introduction and growth of the PVC piping industry in North American markets. Jim Goddard discussed the growth of the polyethylene corrugated pipe industry in North America. Stan Mruk reviewed the history of the Hydrostatic Stress Board (HSB) of the Plastics Pipe Institute (PPI) and its contributions to the development of plastic pressure piping products. Steve Boros discussed the use of long-term hydrostatic strength calculations in the design and development of thermoplastic piping materials and specifically polyethylene piping compounds. Ata Ciechanowski reviewed the role of the National Sanitation Foundation (NSF International) in the development of quality potable water piping products. White Jee discussed the history and the development of American Society for Testing and Materials (ASTM) Standard D3350 and its influence on polyethylene piping compounds.

In the Design section, Michael Pluimer discussed a service life analysis for corrugated high density polyethylene piping for drainage applications. Dr Tim McGrath discussed experience gained in designing storm water chambers to meet American Association of State Highway Officials (AASHTO) specifications. Dr. Richard Brachman presented information on the design and performance of plastic drainage pipes. Dr Tim McGrath reviewed the design development of large thermoplastic chambers for storm water retention systems. Craig Fisher presented considerations in the fabrication of PVC pressure fittings. Amster Howard discussed deflection, lag, lag load and time lag in drainage piping.

In the Testing and Failure Analysis section, John Kurdziel discussed a new stress cracking test method for corrugated high density polyethylene piping. Steve Sandstrum presented a new qualification test method for high density polyethylene butt fusion joining. Craig Fisher discussed the tensile

testing of restrained joint PVC pipes. Flip Sharff discussed failure analysis of plastic pipe and other hydraulic structures. Michael Hayes and Ray Hauser discussed their experiences in investigating CPVC pipe failures. Amster Howard discussed his experience in inspecting large diameter drainage piping.

In the New Materials and Applications section, Frank Volgstadt discussed Polyamide 11 (PA-11) and its use in higher pressure natural gas distribution applications. Richard Wolf discussed Polyamide 12 (PA-12) and its suitability for natural gas distribution piping systems at pressures over 125 psi. Igor Zhadanovsky discussed the use of Polysulfone plastics to replace cast iron piping in steam heating applications.

In the Installation section, Larry Petroff reviewed the specifying of plastic piping products in trenchless technology applications. Steve Gross discussed the use of high performance restrained-joint PVC piping products in directional drilling applications. Richard Botteicher presented a method for calculation installation loads on fusible PVC piping in horizontal directional drilling applications. Dr Ian Moore presented a paper on the axial response of HDPE pipes in directional drilling installations.

The goal of this symposia was to present historical perspectives from industry experts on the successful applications and growth of plastic piping and then to present papers on new materials, new applications and new test methods that are continuing the penetration of plastic piping products in North America and worldwide.

Special thanks to ASTM staff, the authors and presenters and to the reviewers of these papers without whom this STP could not have been completed.

Tom Walsh
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Symposium Chairman and Editor

HISTORICAL REVIEWS

Bob Walker¹

North America's Cinderella Pipe Story: A Look at PVC Pipes' Climb to the Top*

ABSTRACT: Remember the story of Cinderella? Cinderella was too poor and unworthy to ever become a princess. Similarly, when first introduced into the North American pipe market in the 1950s, PVC pipe was admonished by competitors and skeptics as having little chance to succeed. As a substitute for the well-established pipe mainstays of that era—iron, steel reinforced concrete, asbestos cement, and vitrified clay; PVC pipe was initially viewed as having insufficient strength and stiffness to be a viable contender. All of these early views had to be changed. This paper summarily chronicles how the questions and doubts surrounding PVC pipe's performance capabilities were overcome, including the resolution of subsequently raised concerns. Through the consistent treatment of every issue with rational, technical, research based approaches; combined with an extensive in-service record of admirable performance; the use of PVC pipe has grown steadily through more than five decades. As a direct consequence, PVC pipes now account for the majority of all new water and sanitary sewer installations, exceeding the market shares of all the aforementioned established pipe materials combined. PVC pipe's fairy godmother is proud, indeed!

KEYWORDS: PVC pipe, health, environment, safety, viscoelastic, long-term strength, capacity, design basis, corrosion resistance, strength, deflection, crack resistance, root intrusion, joint leakage, tapping, joint restraint, permeation, abrasion resistance, cyclic pressure, sustainable, embodied energy, green, fusion joints, horizontal directional drilling, trenchless

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Introduction

Cinderella is a classic folk tale embodying unjust oppression and unfortunate circumstances that ultimately are changed and the result is remarkable fortune. The word “cinderella” has come to represent anyone whose attributes were unrecognized but eventually was able to achieve success. On Feb. 15, 1950 the Walt Disney Company released their animated feature film of the Cinderella story and it became an instant classic. As a result, most people associate with the Disney version as opposed to the original [1].

Coincidentally, at about the same time Disney was releasing their animated movie classic, plastic pipes manufactured from the plastic poly(vinyl chloride), now commonly referred to as PVC, quietly made their debut in North America.

Chapter One

P.V. Cinderella's Early, Formative Years

Once upon a time on or around 1950, pipe extrusion technology from Europe made its way to North America where several enterprising companies began manufacturing pipes made from a thermoplastic polymer that has come to be known by its initials —PVC. (In the context of this story and the Cinderella analogy, PVC pipe will be appropriately viewed as P.V. Cinderella.) European PVC pipes had demonstrated their abilities to provide durable performance in a number of applications which included buried water distribution and buried sewage collection. However, these first PVC pipes were ahead of their time and lacked the benefits of material and process advancements employed today. It was not until 1950 that the systematic development of modern extrusion technology began. Throughout the 1950s and 1960s effective stabilizers, lubricants and processing aids, together with improved extrusion machinery, all engineered specifically for PVC pipe, were developed [2].

The early installations of PVC pipe were highly successful but hardly a threat to North America's mainstay pipe products for water and sewer, i.e., pipes made from iron, steel, steel reinforced concrete, vitrified clay, and asbestos cement. (In our analogy, these traditional pipe materials were P.V. Cinderella's older and vain stepsisters.) The civil engineers (handsome Prince) largely responsible for selecting pipe appropriate for public and private water and wastewater applications had very little knowledge about PVC (P.V. Cinderella). Moreover, plastics generally were considered to be structurally inadequate for infrastructure construction.

The early advocates of PVC pipe (P.V. Cinderella's fairy godmother) came to understand that many obstacles and misperceptions would have to be overcome if they were going to succeed in fostering acceptance for PVC pipe (i.e., create a lasting eHarmony connection between the handsome Prince and P.V. Cinderella).

Chapter Two

P.V. Cinderella Undergoes Health Screening before Dancing with the Prince

One of the very first, and arguably most critical, issues that had to be addressed was that associated with bringing a new material into our environment. Health officials were especially concerned about any possible adverse effects from materials in direct contact with public drinking water and food supplies. (No Prince wants to end up in an unsafe or unhealthy relationship.)

An impartial, not-for-profit, private research organization known then as the National Sanitation Foundation (now NSF International) was engaged in 1951 by the plastic pipe industry to conduct, "A Study of Plastic Pipe for Potable Water Supplies." The report was completed in 1955 and described a three-year evaluation which examined the suitability of PVC pipes, among others, for underground potable water systems. That study provided public health officials with basic information about the safety of PVC pipes and eventually led to an NSF voluntary PVC pipe certification testing program which began in 1959 and followed by publication of an NSF plastic pipe certification standard in 1965 [3].

In 1984 the U.S. government announced that it wanted to establish a private sector certification program run by a non-profit organization to verify product safety and compliance with national drinking water standards. The program was awarded to a consortium of organizations in 1985 that included NSF, AWWA Research Foundation, Association of State Drinking Water Administrators, and Conference of State Health Managers. In 1988, Standard 61, "Drinking Water System Components—Health Effects," was published for pipes and piping components intended for contact with drinking water. This standard replaced the earlier NSF Standard, and was adopted by the American National Standards Institute (ANSI). The ANSI/NSF Standard includes both regulated and non-regulated substances, and is applicable not only to PVC piping and components, but to all water piping materials [4,5].

Over four decades of testing and certified compliance have verified that PVC pipes do not exhibit significant reaction with even aggressive drinking water and are arguably the safest of all common pipe materials. In addition, tests demonstrate that PVC pipes provide excellent antimicrobial performance and protection against bio-film formation.

Concerns about drinking water becoming tainted from petrochemicals in the soils or groundwater surrounding buried plastic pipes were raised in the 1980s. A number of industry and independently sponsored studies followed to quantify the permeability of the various common pipes and pipe joint materials. The studies revealed that permeability rates vary quite dramatically from one material to the next. When tested, PVC proved to be relatively resistant to permeation. The most recent evaluation of permeability was conducted by Iowa State University with funding from the AWWA Research Foundation [6]. The researchers concluded that PVC pipes present an effective barrier to "permeation of benzene, toluene, and xylenes in either gasoline vapors or gasoline contaminated groundwater at typical contaminated sites" [7].

Having undergone full and complete review by all the independent evaluators of public health and safety, PVC pipe (P.V. Cinderella) was granted admission for use in water and wastewater pipe applications (granted entrance to the grand ball at the palace with authorization to dance with the Prince).

Chapter Three

Time Dependent Properties: Magic Only Until Midnight or for More Than 100 Years?

PVC pipes' inherent viscoelastic nature provides for a stress-strain behavior different from that of traditional pipes made of iron, steel, concrete, and clay. PVC pipes subjected to a sustained internal hydraulic pressure undergo a form of creep, which is time dependent. The result is a non-linear stress-strain relationship. The pressure that causes rupture at 10 000 h is less than the pressure causing rupture at 1000 h and even less than after only 100 h. Higher stress means shorter life, but the converse is also true, the lower the stress the longer the life.

A general lack of familiarity with viscoelastic materials among pipe design engineers, in combination with PVC's relatively lower tensile strength, presented a major obstacle. In order to be accepted and specified for buried water mains and fire protection systems, a method to establish PVC's long-term strength capacity had to be devised.

In November of 1958, a nine-member Working Stress Subcommittee under the Test Methods Committee within the Thermoplastic Pipe Division of the Society of the Plastics Industry was formed. Under the capable leadership of Dr. Frank Reinhart, the Subcommittee worked from 1958 to 1961 and developed a sound engineering basis for evaluating the expected long-term performance of thermoplastic pressure pipe [8]. A large volume of long-term hydrostatic strength testing was then conducted, the vast majority of which was on PVC pipe, and all the collected data was analyzed. This resulted in approval of the first, "Standard Test Method for Obtaining Hydrostatic Design Basis for Thermoplastic Pipe," ASTM D2837 in 1969 [9].

The use of PVC compounds capable of providing predictable, linear, log of time versus log of stress-to-failure behavior per ASTM D2837's requirements, made it possible for pipe design engineers and pipe manufacturers alike to consistently select a design stress low enough to confidently project long-life performance. Perhaps even more importantly, the long-standing elastic design equations familiar to all pressure pipe engineers could be used to design PVC pressure pipes. In order to apply elastic design, engineers needed only to use PVC's long-term design basis strength of 4000 psi, i.e., PVC's ASTM D2837 categorized Hydrostatic Design Basis (HDB), in place of PVC's short-term yield strength (see Fig. 1).

Coincident with this development of a consistent PVC pressure pipe design basis, another engineer, Cecil Rose, with the U.S. Farmers Home Administration boldly pioneered a new design for cost-effective water distribution systems to serve low-income and low-population-density rural areas. In 1956, Mr. Rose advocated using corrosion-free PVC pressure pipe to bring affordable potable

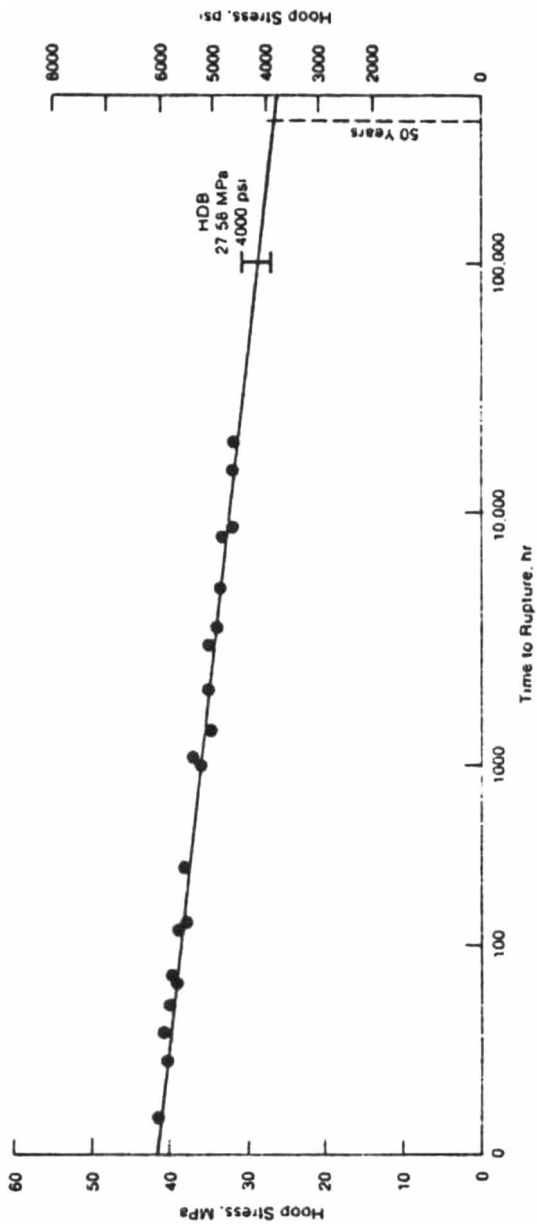


FIG. 1—Typical ASTM D2837 stress-rupture regression line (life line) for PVC pipe [10].

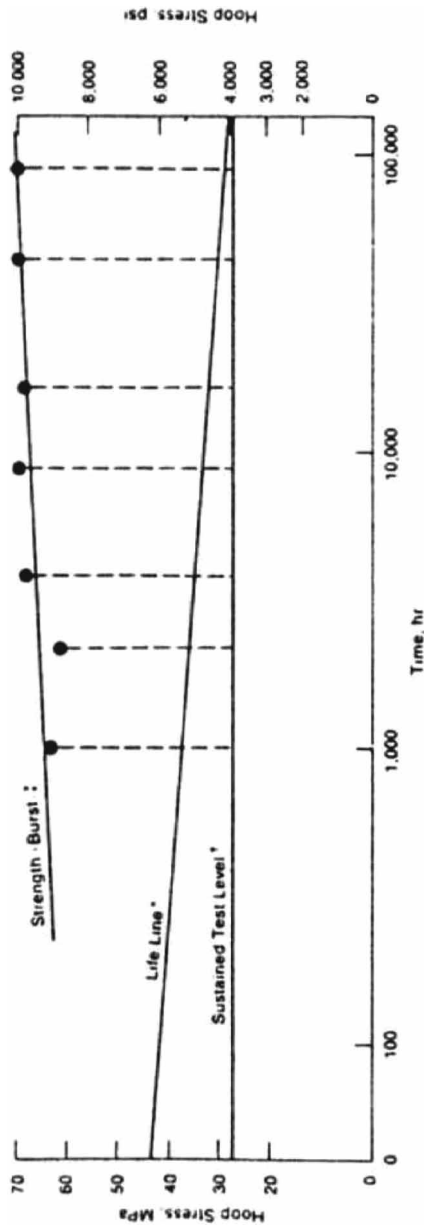


FIG. 2—Strength and sustained stress lines for PVC pipe [11].

water systems to rural America [10]. PVC pipes' lower cost and ease of installation enabled many more communities to be served than otherwise would have been possible. Millions of Americans benefited from his decision.

By 1964 ASTM had published a standard for Pressure Rated PVC pipe, ASTM D2241 [11], and the producers of traditional pipe products (the older stepsisters) were beginning to take notice (of P.V. Cinderella). They were becoming concerned (jealous) as a result of PVC pipes' early and rapidly growing success. Their reaction was the dissemination of untrue negative information about PVC. Their most serious allegation was that PVC pipes lose strength with time when pressurized or under load. The misconception likely originated from misinterpretation of ASTM D2837 test method's downward sloping stress-rupture versus time line.

To determine what happens to the strength of PVC pipe under constant water pressure, pipe testing began in 1970 at the Johns-Manville Corp. Research and Development Center in Denver, CO. All of the PVC pipes tested exhibited an increase in burst strength throughout the entire test period of 10 years. Most impressive was the 22 % increase in burst strength of the sample held at twice its pressure rating for 10 years. The study conclusively showed the accusation that with time PVC pipes lose strength under pressure was incorrect [12] (see Fig. 2).

The American Water Works Association (AWWA) approved their first PVC water pipe standard in 1975. AWWA's PVC pipe standard helped to establish PVC pipe as much more than just a rural pipe product but also a legitimate municipal water distribution pipe and fire protection pipe. Indeed, the water engineering community (the handsome Prince) has made PVC (P.V. Cinderella) the most often installed water pipe material (the most sought after maiden at the ball). By 2004, PVC accounted for 78 % of the combined U.S. and Canadian buried water pipe market for pipe diameters 4 in. and larger [13].

Considering the extensive and widespread use of PVC pipe by the drinking water industry, the American Water Works Association Research Foundation (AwwaRF) and the Australian Commonwealth Scientific and Industrial Research Organization (CSIRO) co-funded a research project to evaluate the sustainable long-term performance of PVC water pipes. The researchers carefully analyzed more than 40 years of available in-service pipe performance information. Their report, published by AwwaRF in 2005, included a performance model that projects a minimum service life of 100 years from PVC pipe when properly designed and installed [14].

Chapter Four

P.V. Cinderella's Rigid Stepsisters Desperately Tried to Keep Her from Attending the Grand Ball

PVC pipes' introduction into the sanitary sewer market lagged slightly behind its use for water distribution. The earliest ASTM standards for PVC sanitary sewer pipes were approved in 1972, i.e., ASTM D3033 [15] and D3034. However, the use of PVC pipe in both drainage and sewer applications in North America dates back to the 1952.

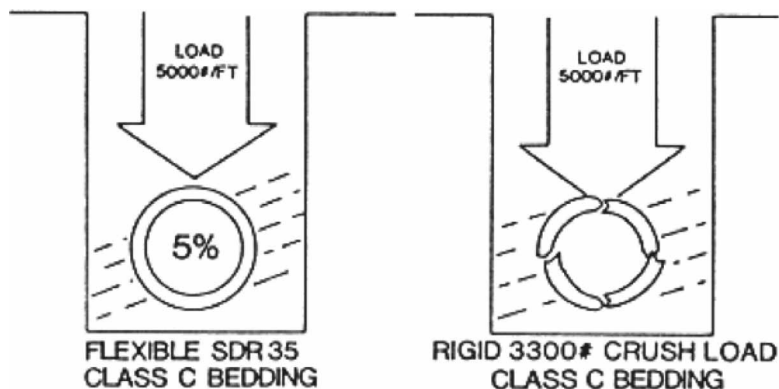


FIG. 3—Load bearing strength of buried PVC sewer pipe exceeds that of rigid pipe [17].

In the 1950s and 1960s vitrified clay pipe dominated the North American sanitary sewer market. For diameters above 30 in., concrete pipes held the largest market share. Clay pipes were highly resistant to chemical attack and had high compression strength. Concrete pipes also possessed high compressive strength but were vulnerable to attack from sulfuric acid generated from sewer gases. The structural strength of both clay and concrete pipes was dependent upon a relatively thick-wall construction.

Concrete and clay pipes' hold on the sanitary sewer market began slipping following passage of major federal water pollution control legislation in 1972 (PL 92-500). That legislation called for stricter controls on wastewater discharges and authorized billions of dollars to assist communities with compliance. To assure that federal and local treatment dollars would be spent efficiently, grant recipients were required to evaluate their sewage collection systems for costly leakage. These evaluations revealed that clay and concrete pipes leaked through joints and through cracks commonly caused by beam breaks in expansive clays, shear failures due to manhole settlement, and shear and beam breaks as a result of shifting soils [16].

Environmental engineers looked for better pipe material options and began lowering the allowable leakage limits for new sewer lines. PVC pipe and PVC fittings were more than up to the challenge. PVC pipes' longer lengths (fewer joints), smoother walls, and much tighter manufacturing tolerances virtually eliminated infiltration leakage. Moreover, when subjected to excess loading or shifting soils, PVC pipes had the ability to flex without cracking or breaking, thereby retaining their leak-free superiority throughout their service life [16].

Pipe utility contractors welcomed PVC sewer pipe. They were tired of having to buy an extra 5 %–15 % to make up for spoilage or breakage during the handling and installation of clay pipe. Contractors also often experienced difficulty in getting installations of new clay and concrete pipes to pass more stringent leakage requirements. PVC pipe afforded them a competitive option that was much easier to handle and install.

However, structural engineers proved to be much harder to convince. They were leery of the lower strength and flexible behavior of PVC pipe. Furthermore,

whole market transformations are rarely easy. Both the clay and concrete pipe industries were large and very well entrenched (no pun intended). Massive resources were committed to try and convince communities that PVC sewer pipes were structurally inadequate and would eventually collapse under trench loads and/or traffic loads. After it became evident that PVC pipes were not collapsing, their negative sales strategy shifted to promoting overly restrictive, post-installation ring-deflection or out-of-round requirements. Specifically, 5 % deflection was misrepresented as “failure” of any buried PVC pipe. Their negative marketing strategy included insisting on post-installation deflection testing, which was a deterrent in the short run because utility contractors were fearful of not being able to pass additional test requirements, but in the long run ended up favoring PVC pipe use.

Concrete and clay pipes are structurally “rigid” pipes. That is, if they do not crack or break, rigid pipes stay round when buried and as such have limited interaction with their surrounding embedment. Conversely, PVC pipes are structurally “flexible.” PVC pipes’ crack resistance is the result of their ability to deflect at least 60 % without cracking. PVC pipes yield or deflect when loaded, thereby developing passive soil support at the sides and around the pipe. This results in a major portion of the vertical load being picked up by the surrounding soil. The outcome is a desirable arching effect. Watkins et al. [17] conducted tests at Utah State University (USU) in 1976 demonstrating that when installed under identical soil conditions, an applied load of 5000 lb/ft (equivalent to a trench depth of 45–50 ft) resulted in structural failure of a rigid pipe with a three-edge bearing strength of 3300 lb/ft, while a PVC sewer pipe with a pipe stiffness of 46 lb/in./in. exhibited very minor deflection of only 5 % (see Fig. 3). The USU comparison addressed the issue of strength adequacy but had not resolved the allegation that 5 % deflection represents failure for a PVC pipe.

In order to objectively evaluate what deflection constitutes failure for PVC pipe, further testing was conducted by Moser and Shupe at USU’s Buried Structures Laboratory in 1980. The PVC pipes performed well structurally (i.e., were able to support increases in loading and to transfer load to the side support soil) right up until inverse or reverse curvature of the pipe was observed. Therefore, while no catastrophic failure was observed, the onset of inverse pipe wall curvature was concluded to be PVC pipes’ structural failure limit. PVC pipes’ critical deflection limit was concluded to be 30 % because inverse curvature was not observed at deflections at or below 30 % [18]. Later application of a conservative safety factor of 4.0 to PVC pipes’ 30 % performance limit resulted in the publication of an Appendix to ASTM D3034 in 1981. The Appendix included a post-installation testing deflection recommendation of 7.5 % (30/4) and defined base inside diameters that appropriately accounted for dimensional manufacturing allowances [19,20].

Communities derived considerable comfort from applying such a conservative post-installation deflection limit to their new PVC sewer pipe installations. Such testing enabled owners to enforce proper installation and has precluded structural difficulties with PVC sewers. The absence of problems spawned the broader acceptance of and a strong preference for PVC sewer pipe. Pipe contractors who were sometimes deterred from using PVC sewer pipe when a 5 % deflection limit was imposed, did not find it unreasonable to comply with the 7.5 % limit.

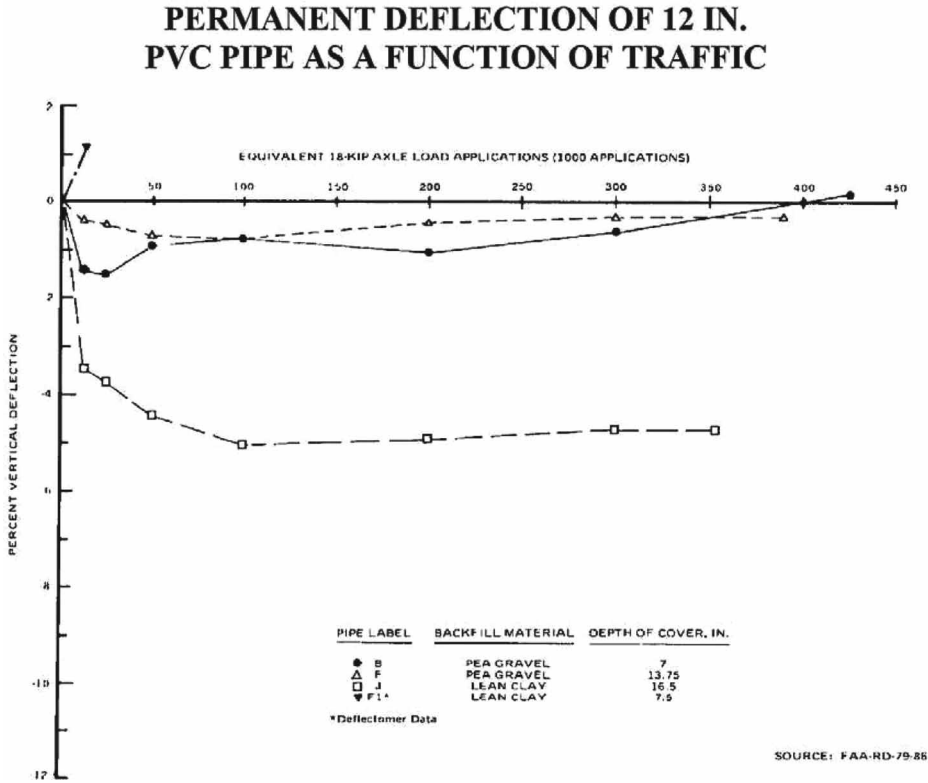


FIG. 4—Permanent deflection of shallow-burial PVC sewer pipe beneath roadway [22].

Testing was also conducted at the USU Buried Structures Laboratory in 1979 to evaluate the root intrusion and leakage performance limits of deflected PVC pipe joints. A 10 lb rock was placed on the spigot end of each joint and adjacent to the adjoining bell to simulate worst-case conditions. Joint leakage did not occur until deflection of the spigot end beneath the rock had reached 43 %, which is above PVC pipes' structural deflection limit of 30 % [21].

The performance research conducted on PVC pipe was effective (like a fairy godmother's magic wand) in debunking the negative sell campaigns that were run by members of the rigid pipe industry (P.V. Cinderella's older rigid and vain stepsisters).

Chapter Five

No Time for P.V. Cinderella to Relax at the Grand Ball

After successfully making it to the grand ball and receiving an enthusiastically favorable response from the Prince, P.V. Cinderella began to look forward to an

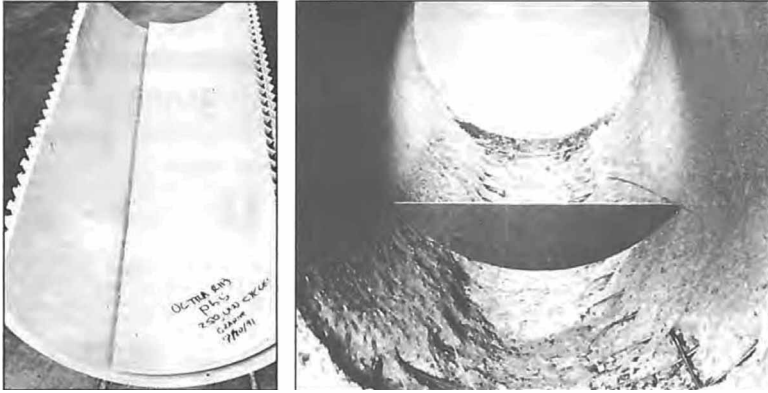


FIG. 5—This profile-wall PVC pipe shows little abrasion wear, whereas the steel reinforced concrete pipe has clearly been completely compromised [34].

enjoyable evening. Regrettably, it did not take long for jealousy to mount among all the other maidens in attendance. They repeatedly tried to trip up P.V. Cinderella by raising new doubts about her. Each doubt had to be systematically addressed; otherwise a less worthy maiden was likely to win over the handsome Prince.

A rumor was circulated that PVC pipe would degrade when exposed to direct sunlight. For a 2-year exposure period beginning in 1977 and concluding in 1979, PVC pipes were placed in exposure racks at locations throughout North America to quantify the effects of natural ultraviolet radiation on PVC pipe. Periodic testing confirmed that neither the pressure capacity (tensile strength) nor ring stiffness (modulus of elasticity) were degraded. Impact strength did decline but remained higher than that of PVC pipes' rigid competitors. The study concluded that the amount of ultraviolet inhibitor (titanium dioxide) used in standard PVC water and sewer pipe is sufficient to allow for 2 years of sun exposure, i.e., shielding from sunlight only needed to be considered for storage periods in excess of 2 years [22].

Another rumor suggested that PVC pipes' would not be able to handle traffic loads, particularly conditions involving shallow depth installations under roadways. Repeated stress variations were known to shorten the lives of many pipe materials through fatigue. The U.S. Federal Aviation Administration Systems Research and Development Service sponsored a series of field tests to evaluate the performance of several plastic pipes under dynamic wheel loadings. The testing commenced in October of 1976 and concluded in March of 1979. All of the testing was done at the U.S. Army Engineer Waterways Experiment Station in Vicksburg, MS and the evaluated burial depths ranged from a mere 7 in. to a maximum of 30.5 in. in order to maximize the traffic loadings. All tests were performed with a flexible bituminous road surface. PVC (DR 35) pipe displayed the best performance among the pipes tested under a range of loadings representative of highway and light to medium aircraft traffic, which included load conditions simulating over 400 000 passes of an 18 kip axle load (see Fig. 4).

As a result, a minimum cover height of 12 in. was recommended for PVC (DR 35) pipe subjected to highway loads of up to 18 kip axle. Under light to medium aircraft loads of up to 320 000 lb gross weight, a minimum burial depth of 2 ft was recommended [23]. Further shallow-burial, traffic load testing was done on larger diameter (24 and 30-in.) PVC sewer pipes. The work was done by the Ontario Ministry of Transportation and published in 1992. As with the earlier testing in Vicksburg, the PVC pipes performed extremely well [24].

The performance capabilities and safety of direct tapping PVC pressure pipes was yet another topic that the PVC pipe industry addressed. The tapping bits that had been used to tap metal and concrete pipes had to be redesigned to safely cut through pressurized PVC pipe. Following the development of the proper cutting or tapping bit designs, testing was conducted by various pipe manufacturers to determine which tapping machines were best suited for tapping PVC pipe and to establish minimum wall thickness limits and diameter limits for direct tapping. Consensus recommendations for direct tapping were published by the Uni-Bell PVC Pipe Association in 1979, which Uni-Bell supplemented in later years through publication of an instructional pocket guide and the production of a tapping training video [25–27]. AWWA addressed the issue of tapping PVC pressure pipe in AWWA M23 and AWWA C605 [28,29].

In order to demonstrate and evaluate the structural integrity of a direct tap, i.e., a service line corporation valve or stop treaded directly in the wall of a PVC pressure pipe, tests measuring the pull out and torque out loads needed to dislodge the corporation valve were conducted [30]. To test the capability of a direct tapped PVC pipe to hold their threaded corporation stops without leakage; 12 tapped samples were subjected to 1.5×10^6 pressure cycles. No leakage, not even a single drop, occurred in any of the pipes tested [31].

Mechanical joint restraints presented another challenge for pressurized PVC pipes. Their effect on pipe performance had not always been properly considered. Therefore, a performance standard was written through a cooperative effort between members of the PVC pipe industry (Uni-Bell) and the manufacturers of mechanical joint restraints, which culminated in 1988 with Uni-Bell's publication of recommended performance requirements followed by approval of ASTM F1674 in 1996 [32,33].

Susceptibility to hydrocarbon permeation in severely contaminated soils was still another question that warranted research and testing. As a result of several evaluations, PVC pipe's resistance to permeation is well understood. (A summary of PVC pipe's resistance to permeation has previously been discussed in "Chapter Two.")

European studies dating back to 1969 have shown that PVC pipes offer excellent resistance to abrasive wear. Nonetheless, the growing popularity of large-diameter profile-wall PVC sewer pipes in North America gave rise to renewed interest in PVC's abrasion resistance capabilities, especially with regard to storm drainage applications. In 1991, comparative abrasion resistance testing was performed at California State University in Sacramento. Using identical crushed rock slurries in parallel tests, the steel reinforced concrete pipe

suffered excess abrasion wear to the point of failure while the PVC pipes experienced minimal abrasion wear [34] (see Fig. 5).

Repeated pressure cycles, such as those associated with sewer force mains and some irrigation pipe systems, presented questions regarding PVC pipes' capability to handle dynamic cyclic loadings. Cyclic pressure testing conducted in the 1970s did not provide an adequate design basis for PVC pressure pipe and most often resulted in overly conservative predictions. Consequently, more research and testing was undertaken at USU, spanning 4 years from 1999 to 2003. The extensive cyclic testing results enabled the researchers to develop a new cyclic design approach for PVC pressure pipe that improved upon all previous approaches [35]. That USU cyclic design approach has been incorporated in the appendix of AWWA C900.

The Prince was more than satisfied by P.V. Cinderella's responses to these various and assorted rumors. As they danced on, the night grew short and the countdown to midnight grew nearer and nearer. P.V. Cinderella remembered to leave before midnight and the Prince looked forward to seeing P.V. Cinderella at the next evening's ball.

Chapter Six

The Prince Learns That P.V. Cinderella is Green

With her fairy godmother's help, P.V. Cinderella attended the ball held on the following evening. The second ball gave the Prince ample opportunity to get to know P.V. Cinderella better. The Prince was quite heavily involved in efforts to promote sustainable infrastructure and curtail global warming. He was very pleased to learn that P.V. Cinderella was environmentally friendly.

As issues regarding environmental protection and sustainability moved higher on public and political agendas, the demand for PVC pipe also increased. Indeed, the PVC pipe industry owes much of its success to environmentalists and the environmental movement. More importantly, our environment has benefited immensely as a result of the widespread use of PVC pipe [16].

PVC pipes' rise to prominence was rooted in PVC's inherent suitability for handling both the physical/mechanical and chemical/environmental conditions associated with our essential water and wastewater pipe infrastructure systems. PVC pipe's excellent resistance to chemical attack and immunity to both galvanic and electrochemical corrosion meant that when designed properly, PVC pipe would perform for hundreds of years. PVC pipes are not damaged by corrosive soils and are not affected by sulfuric acid in the concentrations found in sanitary sewer systems. As a result, no linings, coatings, or cathodic protection are required when PVC pipes are used [36].

For a pipe to be considered sustainable it needs to be produced with minimal adverse environmental impact and minimal carbon footprint, provide for low life cycle cost, be maintenance-free or require minimal maintenance, and be able to be economically recycled at the end of its useful life [37].

Many people are under the false impression that plastic pipes use more natural resources in the form of fossil fuels than traditional pipe materials. In fact,

the reverse is true, because most of the heat energy required for the production of ductile iron, steel, copper, and aluminum comes from oil or coal. Metal pipes require very high temperatures to produce and thus are extremely energy intensive [37].

An Australian (CSIRO) study compared the embodied energy of various pipe products and concluded that, "Within all scenarios, virtually all the various forms of PVC pipes produced lower embodied energy results than any other piping material." Embodied energy is defined as "the quantity of energy required by all of the activities associated with a production process, including the relative proportions consumed in all activities upstream to the acquisition of natural resources and the share of energy used in making equipment and in other supporting functions, i.e., direct energy plus indirect energy." The study considered the energy required to manufacture the raw materials, transport of the raw materials, the pipe manufacturing process, as well as the embodied energy implications of using different piping solutions to achieve a similar hydraulic performance over a fixed length [38]. The latter is important because PVC pipes are more flow efficient as a result of their smoother interior surfaces.

Thus, PVC pipe use lowers carbon dioxide emissions that contribute to global warming. PVC pipes are generally easier to cut in the field and can be joined to standard appurtenances (valves, fittings, hydrants, etc.) without the need for special connections or adaptor couplings. Properly assembled PVC pipe joints are leak-free and PVC pipe's strain ability resists cracking. By and large, PVC pipes also require little or no maintenance when designed and installed correctly.

Furthermore, PVC is produced by combining ethylene (derived from either natural gas or petroleum) and chloride from salt (sodium chloride). As a result, only 43 % of PVC comes from a non-renewable resource in the form of fossil fuels. Hence, PVC pipe is an excellent choice from the standpoint of non-renewable resource conservation [39].

Consequently, the Prince's regard for P.V. Cinderella increased during the second ball. But P.V. Cinderella had lost track of time and ending up having to leave at the stroke of midnight, losing one of her glass slippers in the process. The Prince retrieved the slipper and vowed to find and marry the girl to whom it belonged.

Chapter Seven

The Search Leads to a Very Happy Ending

In this Cinderella story the Prince did not have to search far and wide to find P.V. Cinderella because PVC pipe has become today's most widely utilized pipe material in the United States and Canada for transporting fluids, especially in drinking water distribution systems and wastewater collection systems [13]. Owner and contractor familiarity with PVC pipe is well established. PVC pipes are also often selected for sewer force mains, agricultural and turf irrigation piping, storm water drainage, and a variety of industrial and plumbing pipe applications.

While the combination of outstanding performance and value for more than five decades has certainly been essential to sustaining PVC pipe's market growth, this story was written to highlight the fact that a tremendous amount of testing and research has taken place—and we only hit the highlights. The time and money invested in properly addressing the issues raised by PVC pipe's many skeptics, competitors, and customers in a technical, scientific manner has proven to be invaluable. Certainly the benefits that have been, and continue to be, derived from the millions of miles of PVC pipe in service far exceed the costs and resources invested in the industry's success.

Moreover, this happy ending story is not ending. New technological developments have enabled PVC pipe to expand in to several emerging applications for which PVC pipes are well suited. These include reclaimed/recycled water systems, trenchless directional drilled installations, and pipeline rehabilitation.

The time has come to replace and repair our aging pipe infrastructure in ways that use less energy and inflict substantially less harm to our environment. There is a growing trend toward minimally intrusive trenchless methods, particularly in congested urban environments. Horizontal directional drilling, often referred to as HDD, has increased from 12 operational units in 1984 to thousands of units today [40]. This rapid growth has been driven by technological advancements in precision drilling equipment in conjunction with rising costs involved with construction in congested urban areas. Other contributing factors are the increased awareness of the social costs of open cut construction and increasing environmental regulations, especially for pipes crossing rivers or passing through wetlands or other environmentally sensitive areas [40].

The joining of PVC pipe via thermal butt fusion has been developed and has been highly successful in waterworks projects, particularly in trenchless applications and installations. The technique allows for the joining of multiple pipe sections into a continuous, gasket-less length of PVC pipe that can be pulled in to place. PVC pipe joined by butt fusion facilitates installation through alignments created by a directionally drilled path or through an existing pipeline, or installations involving pipe bursting, and eliminates the need to enlarge the bore hole to accommodate the external protrusions associated with bell ends, couplers, or mechanical restraints.

Two advancements have made the field fusion of PVC pipe possible. The first was the development of a specific material formulation, or recipe, capable of consistently producing high-strength fusion joints. The formulation is fully compliant with the industry's established generic formula requirements for pressure-rated PVC pipe. The second advancement was the development of the particular conditions and procedural steps for reliable in-the-field fusion of PVC pipe ends. The result has been leak-free joints that consistently provide strength equal to that of the pipe barrel.

So P.V. Cinderella's fairy godmother accomplished her mission. The Prince married P.V. Cinderella and together their happiness is continuing to grow with time or, to put it more succinctly, they are living happily ever after.

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A Brief History of the Development and Growth of the Corrugated Polyethylene Pipe Industry in North America*

ABSTRACT: From 1966 to 2009, the corrugated polyethylene pipe industry has grown from its first introduction to greater than a 2.5×10^9 industry in North America and from 4 in. (100 mm) only to a product diameter range of 2 in. (50 mm) through 60 in. (1500 mm). Markets have changed and grown as well, from predominantly agricultural drainage applications to housing, commercial development, transportation, mining, industry, forestry, storm sewer, sanitary sewer, turf drainage, and stormwater treatment. Increases in diameters and expansion of markets have led to corresponding changes in manufacturing processes and development of appropriate standards and specifications.

KEYWORDS: pipe, polyethylene, corrugated pipe, corrugated polyethylene pipe, culvert, storm sewer, land drainage, pipe history, pipe testing, pipe specifications

Introduction and History

Polyethylene was accidentally synthesized by Hans von Pechman, a German chemist, in 1898. Eric Fawcett and Reginald Gibson, of imperial chemical industries (ICI), again by accident, produced the first industrially practical polyethylene in 1933 in England, though it was not until 1935 that Michael Perrin, also with ICI, developed a reproducible high-pressure synthesis for low-density polyethylene (LDPE). Commercial production based on this work began in

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1939. High density polyethylene (HDPE) was first produced by Robert Banks and J. Paul Hogan of Phillips Petroleum in 1951.

LDPE pipe was first manufactured in 1945, primarily for small diameter water service piping. HDPE pipe was first manufactured in 1955. HDPE has a higher design hoop stress and was manufactured in larger diameters than the earlier LDPE pipe. These pipes were all smooth wall products.

The first corrugated plastic pipe produced for land drainage was 50 mm (2 in.) diameter and was manufactured from either polyvinyl chloride (PVC) or polyethylene (PE) in Europe in 1961. Small diameter (2 and 3 in.) (50 and 75 mm) corrugated electrical duct was already being manufactured by Haveg Industries, Inc., of Wilmington, DE.

The first corrugated polyethylene pipe commercially produced in the United States was made in Middletown, DE, on July 3, 1967, by Advanced Drainage Systems, Inc., what was then basically a 4 men company. The first pipe was 4 in. (100 mm) diameter. The intended market was agricultural drainage to increase crop yields, replacing 12 in. (300 mm) long, 4 in. (100 mm) diameter clay tile, which dominated the market at that time, but was cumbersome and costly to install. Four 4 inches (100 mm) diameter was chosen because the vast majority of the clay tile used by the agriculture industry was 4 in. (100 mm). Changing material use from clay tile to corrugated polyethylene was enough of a hurdle without also trying to sell a reduced diameter.

The first significant order for this new pipe, two full truckloads, was not for agriculture, but for draining a golf course near Louisville, KY. A second two truck order was delivered to Musser's Nursery in Indiana, PA. Then a farm drainage contractor in southwest IA ordered 20 rail cars of 4 in. (100 mm) pipe at 20 000 ft (6100 m) per rail car (a 400 000 foot order) for agricultural drainage applications. The ability to coil this pipe into lengths of several thousand feet and to then install it with high-speed trenchers or utility plows spurred significant growth and considerable cost savings to the farmer. By 1978, the clay tile business was obsolete, with a number of the more progressive clay tile companies by then manufacturing corrugated PE pipe.

The agriculture market dominated the business throughout the 1960s and 1970s, though use around homes for drainage and septic leach fields and around commercial buildings grew to about 40 % of the business. The demand for larger diameters drove manufacturers to produce 5 in. (125 mm), 6 in. (150 mm), 8 in. (200 mm), 10 in. (250 mm), and, by 1978, 12 in. (300 mm). This increased diameter range also increased the available markets.

The first Department of Transportation (DOT) projects were installed as highway underdrains in the early 1970s: by the Iowa DOT on Interstate 80 and by the Georgia DOT on Interstate 20 [1,2]. The Georgia DOT was the first to include corrugated polyethylene pipe in their standard specifications, referencing the American Society of Testing and Materials International (ASTM) specification developed for agricultural drains, ASTM F405. The I-20 project in GA had 192 000 linear ft (58 350 m) of 4 in. (100 mm) underdrain pipe installed at an average rate of 2000 ft (608 m) per 8 h a day per crew in the winter of 1974 and spring of 1975. The Federal Highway Administration, Offices of Research and Development issued "Implementation Package 76-9, Slotted Underdrain

Systems" in June of 1976 detailing the use, installation requirements, limitations, and performance of underdrain materials, including corrugated steel pipe, corrugated PE pipe, and slotted PVC pipe [3].

The Federal Highway Administration followed this report with a structural test conducted at the Bureau of Reclamation laboratory in Denver utilizing a 7×7 ft ($2.1 \text{ m} \times 2.1 \text{ m}$) steel soil cell under their Baldwin compression testing machine titled, "Structural Response of Selected Underdrain Systems" in the summer of 1976. The pipe diameter tested was 6 in. (150 mm). In the description of test 9, "A slotted, 6-inch, PE pipe was tested in 12 in. of loosely shoveled concrete soil. This pipe had been tested in Part I of the testing program and had lain in the outdoors for two years unprotected with no obvious damage."

The first major airport drainage project was for the Jacksonville Florida airport in 1976. This project utilized corrugated polyethylene pipe for underdrain along and across the runway in fabric wrapped trenches with recycled concrete as aggregate backfill. The rebuilding of the runway took 92 days, 58 days ahead of schedule [4]. Since then, corrugated polyethylene pipe has been used for underdrain, stormwater collection, and water treatment applications at many airports throughout the United States, including Atlanta Hartsfield, Dallas—Fort Worth, Denver International, Pittsburgh, and Chicago O'Hare.

In late 1979, 15 in. (375 mm) pipe was added to the available diameters.

By 1980, the North American corrugated polyethylene pipe industry had grown to over \$150,000,000 per year in total sales and about 58 % of that was still for the agricultural market.

Growth in volume and applications accelerated after 1981, with the introduction of 18 in. (450 mm) and 24 in. (600 mm) pipe. In September of 1981, the Ohio Department of Transportation installed the first known corrugated polyethylene crossdrain culvert under a state highway. The site is located in southeastern Ohio and this pipe replaced a failing culvert that had collapsed due to attack from the low pH ($pH < 4$) flow through it. At this site, the Ohio DOT had been replacing other types of pipe every 2–5 years. This PE crossdrain is still in service after 28 years and appears unchanged. In relatively few years, state DOT maintenance departments were using corrugated PE pipe extensively to replace pipe of other materials in areas where corrosion was a problem (Fig. 1).

One of the seminal technical developments in the industry, at least as it applied to the U.S. markets, was a decision made by Advanced Drainage Systems, Inc., in March of 1983, to promote and produce a variable pipe stiffness, with nominal pipe stiffness decreasing with increasing diameters. This was a substantial deviation from the direction PVC sanitary sewer pipe was taking, with a constant pipe stiffness of 46 pounds per inch of sample length per inch of deflection per ASTM D2412. But it was not all that radical, with corrugated steel pipe having a nominally lower pipe stiffness as diameters increased and with corrugated aluminum pipe having much lower values throughout the diameter range. It was also analogous to the stiffness values in ASTM F894, which hid the decreasing values by establishing a Ring Stiffness Constant (RSC) based on testing at 4 times the loading rate of ASTM D2412 (2 in./min rather than 0.5 in./min (50.8 mm/min rather than 12.7 mm/min) and determining the stiffness at 3 % deflection instead of 5 % in D 2412. The actual stiffness values selected are



FIG. 1—Ohio DOT Installation of 24 in. (600 mm) diameter culvert after 27 years in acid mine run-off.

shown in Table 1, below, along with comparable corrugated metal pipe stiffnesses.

The corrugated metal pipe (CMP) values are based on standard sinusoidal 2-2/3 in. $\times$ 1/2 in. corrugations in the standard gages for those diameters. These corrugated HDPE pipe stiffness values were generated when the largest diameter manufactured in the United States was 24 in. (600 mm) diameter. They represent a decreasing flexibility factor as diameters increase, while the CMP industry promoted a constant maximum flexibility factor for their pipe: 9.5×10^{-2} for corrugated aluminum pipe (as flexibility factor goes down, stiffness goes up). The

TABLE 1—Comparative pipe stiffness (in pounds per inch of sample length per inch of deflection at 5 % deflection [pii]).

Diameter (in.) (mm)	Corrugated steel pipe	Corrugated aluminum pipe	Corrugated HDPE pipe	Current AASHTO M 294-08 (kPa) (pii)
12 (300)	145	48.3	45	345 (50)
15 (375)	104	34.7	42	290 (42)
18 (450)	79	26.4	40	275 (40)
24 (600)	51	17.1	34	235 (34)
30 (750)	41	13.8	28	195 (28.3)
36 (900)	31.5	10.5	22	150 (21.75)
42 (1050)	30	10.0	20	140 (20.3)
48 (1200)	24	8.0	17.5	125 (18.1)
60 (1500)	20.1	6.7	14	95 (13.78)
72 (1800)	15.3	5.1	11	...
84 (2100)	13.7	4.6	10	...

flexibility factor of the corrugated HDPE 12 in. (300 mm) pipe is 9.5×10^{-2} , but the flexibility factor for the 60 in. (1500 mm) pipe is 6.3×10^{-2} . This information was published in Transportation Research Record 903 authored by Watkins et al. as a part of a broader paper that includes loading tests on 15 in. (375 mm), 18 in. (450 mm), and 24 in. (600 mm) pipe with both soil cell test results and shallow bury live load tests.

This variable pipe stiffness approach permitted the design and marketing of very large diameter pipe at competitive costs to competitive materials, something that a constant pipe stiffness of 46 pii (320 kPa) would not have permitted. These minimum pipe stiffness values are required, unchanged from the 1982 recommendations except for the increase of 12 in. (600 mm) pipe stiffness from 45 pii (310 kPa) to 50 pii (345 kPa), by the current American Association of State Highway and Transportation Officials (AASHTO) M 294 specification and in ASTM F2306 and F2648.

The Canadian industry continued the practice of maintaining a constant 46 psi (317 kPa) pipe stiffness throughout the diameter range. The impact of this decision was to limit the maximum diameters that could be manufactured at competitive prices to conventional materials. Generally, the largest diameter pipe produced to Canadian standards was 36 in. (900 mm).

Polyethylene resins available to the industry have changed significantly as the industry grew. In the early years, the primary resins used were "bottle grade" materials, which could be variable in processing performance from lot to lot. Today, the major resin companies all manufacture resins specifically tailored for this industry and these pipe applications. These materials have very consistent material properties and a high stress crack resistance.

The next big increase in applications and use came with the development of the manufacturing technology to make corrugated polyethylene pipe with a smooth interior wall in 1987. Developed primarily to improve the flow capacity of the pipe by substantially lowering the Manning's "n" value, the new pipe also had substantially increased longitudinal stiffness, making it easier to install. The industry now had a product that could compete with other smooth interior pipe types.

At the same time, larger diameters were being developed, with 30 in. (750 mm) and 36 in. (900 mm) introduced in 1987, 42 in. (1050 mm) and 48 in. (1200 mm) manufactured in 1991, and with 60 in. (1500 mm) produced in 1998. In those diameters, only smooth interior pipe has been manufactured and marketed.

With each increase in diameter, Moser et al. of Utah State University conducted the soil cell tests on those new diameters as reported in TRR 903 [5–7]. These tests were open to Department of Transportation engineers and others, and at various times, at least five DOTs were represented at the test site in Logan, UT. Each diameter or significant wall design change was tested in three different density fills, 95 %, 85 %, and 75 % Standard Proctor Density (SPD). The most important information gained from these tests was the relative performance of this pipe to other pipe types previously tested the same way in the same test facility. These comparisons demonstrate that a properly designed pipe wall profile can outperform a much stiffer pipe with a less stable wall



FIG. 2—Utah State University: soil cell full, loading beams down, and load being applied to soil surface over 60 in. (1500 mm) pipe.

design. Most of this testing is reported in the textbook “Buried Pipe Design” by Moser and Folkman, pp. 471–503 (Fig. 2).

Also, with each increase in diameter, test installations were placed under pavement by various Departments of Transportation, normally by their maintenance forces. For instance, the first 24 in. diameter crossdrain was installed by the Ohio DOT [8]. The first two 48 in. (1200 mm) installations were put in by the Ohio DOT and by PennDOT. The first 60 in. installation put in by a DOT was installed by the Missouri DOT. All of these pipes are still in place and performing well.

In 1987, with the cooperation of the Pennsylvania Department of Transportation and Mashuda Construction and technical assistance from Dr. Ernest Selig of the University of Massachusetts, Advanced Drainage Systems had a 24 in. (600 mm) pipe installed under I-279 north of Pittsburgh, PA, with a maximum fill height of 100 ft (30.5 m). This pipe has been the subject of thousands of pages of reports, including a full inspection and corresponding report after 20 years. The total pipe length is 576 ft, with 220 ft being smooth interior and the remainder being corrugated inside and out [9–12]. There has been a great deal learned, or confirmed, from this study, including the following:

- (1) Under very high fills and corresponding loads, the HDPE compresses slightly under wall thrust. In this case, the maximum hoop compression was 1.9 %.
- (2) The hoop compression and subsequent hoop shortening, combined with the vertical deflection creates a substantial soil arch over the pipe. In this case, the vertical soil pressure at the crown of the pipe is about 23 % vertical soil column weight.
- (3) Deflection and shortening stabilized in a relatively short period of time: less than one year after completion of the fill.

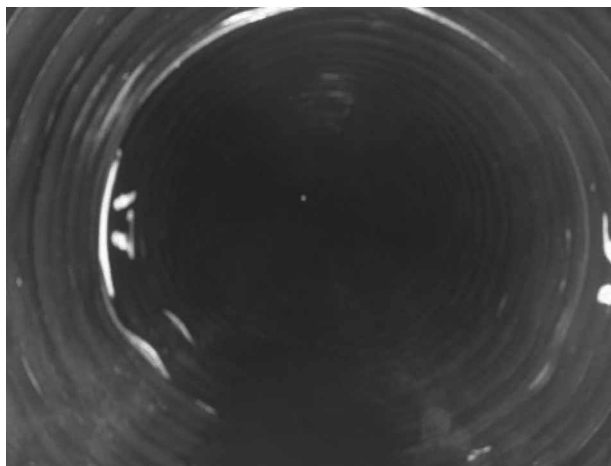


FIG. 3—24 in. (600 mm) Type S under I-279 at about 80 ft (25 m) of fill.

- (4) Anti-oxidant levels within the HDPE material have remained much higher than anticipated after 20 years of service, with very little reduction in OIT or OITemp values. These values represent the thermal stability of the material as tested in accordance with ASTM standards.
- (5) Material properties, specifically tensile strength at yield and flexural modulus, have not changed over 20 years of service [13–15].

The Pennsylvania Deep Burial Study confirmed that this type of pipe could withstand substantial loads for long periods of time without failure. It has become a very convincing proof for many specifying engineers [16–20] (Fig. 3).

In 1999, Ohio University installed six runs 900 ft (274 m) long of thermoplastic pipe in an embankment condition with 20 ft (6 m) and 40 ft (12 m) of fill in a research project funded by the Ohio Department of Transportation, the Pennsylvania Department of Transportation, the Federal Highway Administration, Advanced Drainage Systems, Inc., and Lane Enterprises, and product from Contech Construction Products, and Lamson & Sessions, Inc. Pipe diameters and quantities included 3600 ft (1095 m) of 30 in. (750 mm) diameter pipe, 900 ft (274 m) of 42 in. (1050 mm) diameter pipe, and 900 ft (274 m) of 60 in. (1500 mm) diameter pipe. Backfill materials included gravel or sand and compaction levels were 86 % SPD, 90 % SPD, or 96 % SPD. Bedding thickness was also varied on two of the runs. Two of the 30 in. (750 mm) runs were profile wall PVC pipe and the remaining runs were corrugated HDPE pipe. There have been no signs of structural distress in any of the pipes in this study. Sensor readings that represent pipe performance and soil-structure interaction stabilized within three months or less after construction was completed. Circumferential shortening in the HDPE pipe was -0.1% to less than 1% . The vertical arching factor for the pipe buried 40 ft (12 m) deep ranged from 0.65 to 0.28 [21].

After the addition of the smooth interior and the development of larger diameters, the changes in pipe joint performance have significantly changed the

industry and the markets available to these products. In the 1970s and 1980s, the typical pipe joints were split external bands or couplers, which held the pipe together well, but which leaked. In-line bell and spigot designs were developed with a bell outside diameter equal to the pipe outside diameter. Initially, with a gasket, this joint provided a highly leak resistant joint that precluded the migration of soil fines through the joint. This was followed by the reinforcing of the joint to provide a “watertight” joint equivalent to current sanitary sewer joints and watertight per ASTM D3212 at 10.8 psi (75 kPa) in a laboratory test.

Specification History and Development

The growth of any widely used pipe product is directly impacted by the development of standard specifications. In the United States, ASTM and the AASHTO are the primary developers of pipe related material standards. Key to the corrugated polyethylene pipe industry’s growth have been shown in Table 2.

The development and passage of ASTM F405, Standard Specification for Corrugated Polyethylene (PE) Pipe and Fittings, followed earlier standards developed by the Soil Conservation Service of the U. S. Department of Agriculture and the United States Bureau of Reclamation, whose specifications were focused primarily on land drainage. F 405 was referenced by Departments of Transportation, by civil engineering consulting firms, and by golf course designers. It began the expansion of the use of this type of pipe in applications other than land drainage.

ASTM F449, Standard Practice for Installation of Corrugated Polyethylene pipe for Agricultural Drainage or Water Table Control, passed and published in 1976, offered installation guidance for agricultural drainage and other water table control applications. This was a much needed standard, in large part because installation of this pipe was significantly different from earlier experience with the clay tile, with high-speed machines (to the point that installing 10 000 ft (3 km) per day of 4 in. (100 mm) pipe was not uncommon with a single machine and crew), and the reduction in hand work and pipe placement.

ASTM F481, Standard Practice for Installation of Thermoplastic Pipe and Corrugated Pipe in Septic Tank Leach Fields, reflected a totally different application and market, the septic leach field, where instead of removing water or fluids from the ground, the effluent was being put back into the ground. Again, clay tile had dominated this market with open joint pipe. The corrugated PE pipe was connected with couplings or by bell and spigot joints and the pipe had/has round perforations in the lower half of the pipe. This specification provided state and local health departments with regulatory responsibility over these installations from an installation guide that they could refer. This application also gave birth to the “stripe,” which was used to locate the top of the pipe so that the perforations were always in the proper orientation. Many manufacturers paint the stripe on the pipe, but some extrude a colored stripe directly into the PE parison as it leaves the die head and before it moves into the forming molds.

ASTM F667, Standard Specification for Large Diameter Corrugated Polyethylene Pipe and Fittings, initially passed in 1980, reflected the increasing diameters being produced by the industry, covering 8 in. (200 mm), 10 in. (250 mm),

TABLE 2—ASTM standards.

ASTM standards		
ASTM designation	Title	Year approved
F405	Standard Specification for Corrugated Polyethylene (PE) Pipe and Fittings	1974
F449	Standard Practice for Installation of Corrugated Polyethylene pipe for Agricultural Drainage or Water Table Control	1976
F481	Standard Practice for Installation of Thermoplastic Pipe and Corrugated Pipe in Septic Tank Leach Fields	1976
F667	Standard Specification for Large Diameter Corrugated Polyethylene Pipe and Fittings	1980
F2306	Standard Specification for 12 to 60 in. [300 to 1500 mm] Annular Corrugated ProfileWall Polyethylene (PE) Pipe and Fittings for Gravity-Flow Storm Sewer and Subsurface Drainage Applications	2005
F2648	Standard Specification for 2 to 60 inch [50 to 1500 mm] Annular Corrugated Profile Wall Polyethylene (PE) Pipe and Fittings for Land Drainage Applications	2007
D7001	Standard Specification for Geocomposites for Pavement Edge Drains and Other High-Flow Applications ¹	2004
F2433	Standard Test Method for Determining Thermoplastic Pipe Wall Stiffness	2005
D1248	Standard Specification for Polyethylene Plastics Extrusion Materials for Wire and Cable	1952
D3350	Standard Specification for Polyethylene Plastics Pipe and Fitting Materials	1974
F2136	Test Method for Notched, Constant Ligament-Stress (NCLS) Test to Determine Slow-Crack-Growth Resistance of HDPE Resins or HDPE Corrugated Pipe	2001

12 in. (305 mm) 15 in. (375 mm), 18 in. (450 mm), and 24 in. (600 mm). The pipe covered by this specification is corrugated inside and outside. This pipe is usually manufactured in 20 ft (6.08 m) lengths, though many manufacturers will provide coil of up to 18 in. (450 mm) diameters for installation by high-speed trenchers. Applications remain largely agricultural land drainage for these pipe sizes, though they have been used by transportation agencies for slope drains, where the corrugations serve to reduce the flow velocity.

ASTM F2306, Standard Specification for 12 to 60 in. [300 to 1500 mm] Annular Corrugated Profile- Wall Polyethylene (PE) Pipe and Fittings for Gravity-Flow Storm Sewer and Subsurface Drainage Applications, approved in 2005, increased the diameters included in ASTM standards, from 24 in. (600 mm) in F 667 to 60 in. (1500 mm). It also expanded the applications covered by ASTM standards. The pipe included in this standard has a smooth interior wall and a

corrugated exterior wall. Watertight joints meeting ASTM D3212, Specification for Joints for Drain and Sewer Plastic Pipes Using Flexible Elastomeric Seals, are included in the standard as one of three pipe joint options. The minimum pipe stiffness requirements follow the recommendation made in 1982 and included in Table 1 for corrugated polyethylene pipe.

ASTM F2648, Standard Specification for 2–60 in. (50–1500 mm) Annular Corrugated Profile Wall Polyethylene (PE) Pipe and Fittings for Land Drainage Applications, provided the opportunity for the industry to produce a pipe for less rigorous applications than ASTM F2306 with slightly reduced properties [22]. This provided a lower cost alternative for installations with limited loading requirements.

ASTM D7001, Standard Specification for Geocomposites for Pavement Edge Drains and Other High-Flow Applications, was developed to cover pipe-like structures that were planar and generally included a geotextile wrap as a filter. These products were initially used to provide pavement edge drainage, generally at the pavement and shoulder seam. Use of this type of product has expanded to include turf drainage: for major athletic fields (such as Turner Field in Atlanta, the Miami Dolphins stadium, the Olympic Stadium in Sydney, Australia, and many others) and for golf course drainage, especially golf greens.

ASTM F2433, Standard Test Method for Determining Thermoplastic Pipe Wall Stiffness, was totally research based. Most of the early corrugation or profile wall designs were intended to provide the required pipe stiffness while using a minimum of material and to be readily manufacturable. As diameters increased, this led to designs that were stiff but unstable in bending. This test was developed to determine the performance of a pipe wall design under combined bending and compression loads. The research behind it was done at California State University—Sacramento by Dr. Lester Gabriel and James B. Goddard. In the research, it was determined that the test could provide the following:

- (1) the stiffness of the wall section;
- (2) a comparison of various wall designs;
- (3) a comparison of different materials in the same wall design;
- (4) a time-dependent pipe wall stiffness and effective relaxation modulus.

The test could be a QA/QC test for very large diameter pipe that could not efficiently be tested in accordance with ASTM D2412. From this work, larger diameter pipe walls were designed that were both very efficient and very stable. Some 42 in. (1050 mm) and larger diameters are produced today based on this standard that are stable through greater than 40 % deflection. And looking at the Utah State University soil cell tests, the stability of the pipe wall can have more influence on pipe performance in the ground than does the pipe stiffness. ASTM F2433 is a tool to assist in the design of larger diameters and different wall profiles.

No discussion of the development of standards that impacted the growth of the corrugated polyethylene pipe industry would be complete without a review of the PE resin specifications and their changes over the corresponding time period. The earliest corrugated polyethylene pipe standards, ASTM F405, ASTM F667, and AASHTO M 252, referenced ASTM D1248. ASTM D1248 was not

limited to wire and cable applications initially and was used by the PE pipe industry for many years, for pressure pipe as well as non-pressure pipe. The change to ASTM D3350 as the materials reference for corrugated polyethylene pipe began with the development of AASHTO M 294, Corrugated Polyethylene Pipe, 300 to 1500-mm Diameter, and other standards that followed. ASTM F405 and F667 were changed to drop the D1248 reference and replace it with an ASTM D3350 reference in 2006. Utilizing the ASTM D3350 cell classification system permits more precise definitions of material properties. Specifically, the cell class required in AASHTO M 294 and ASTM F2306 is 435400 C, which limits the material density to 0.947–0.955 gm/cc; the melt index to <0.4 to 0.15 gm/10 min; the flexural modulus to 110 000–160 000 psi (758–1103 MPa); the tensile strength at yield to 3000–3500 psi (24–28 MPa); the carbon black percentage to a minimum of 2 %. The cell class for Environmental Stress Cracking is not used but is replaced by ASTM F2136, Test Method for Notched, Constant Ligament-Stress (NCLS) Test to Determine Slow-Crack-Growth Resistance of HDPE Resins or HDPE Corrugated Pipe, which was specifically developed to test non-pressure pipe resins and/or test specimens taken directly from the pipe wall.

The development and continuous improvement of these ASTM standards has contributed greatly to the growth of this industry and the acceptance of these pipe products. The changes in these standards over time have reflected the maturing and growth of the industry. These specifications give the user and specifier confidence that they are using materials that meet the quality standards required to perform as desired in varied applications.

The AASHTO specifications have also had significant impact on the industry's growth. AASHTO specifications are referenced by the State Highway Departments, the District of Columbia Transportation Department, the Puerto Rico Highway Department, the Federal Highway Administration, the Federal Aviation Administration, the Army Corps of Engineers, and other government agencies. These AASHTO pipe specifications shown in Table 3 are developed by the AASHTO Subcommittee on Materials, and only State Materials Engineers can vote on them for passage or changes.

AASHTO M 252, Corrugated Polyethylene Drainage Pipe, was first passed and published by AASHTO in 1976 and was intended, primarily, for underdrain or subdrain applications. Initially, the standard just included 4 in. (100 mm) and 6 in. (150 mm) pipe, but as available diameters increased, they were added to M 252, until, ultimately diameters included through 15 in. (375 mm).

TABLE 3—AASHTO standards.

AASHTO standards		
AASHTO designation	Title	Year approved
M 252	Corrugated Polyethylene Drainage Pipe	1976
M 294	Corrugated Polyethylene Pipe, 300 to 1500-mm Diameter	1986

TABLE 4—AASHTO pipe standards.

AASHTO standard	Polyethylene pipe
Subcommittee on Materials	AASHTO M 252 AASHTO M 294
Subcommittee on Bridges and Structures—Design	AASHTO LRFD Bridge Design Specifications—Section 12
Subcommittee on Bridges and Structures – Construction	AASHTO LRFD Bridge Construction Specifications Section 30
National Transportation Product Evaluation Program	NTPEP for Thermoplastic Pipe

In 1986, AASHTO M 294, Corrugated Polyethylene Pipe, 300 to 1500-mm Diameter, was passed and published. Initially, the M 294 specification included on 12 in. (300 mm) diameter through 24 in. (600 mm) diameter, with the 12 in. (300 mm) and 15 in. (375 mm) being removed from M 252 at that time. As available diameters increased and the appropriated testing on those diameters was done the new sizes were added to the standard.

AASHTO goes beyond materials standards and has design standards, construction standards, and a quality assurance program as outlined in Table 4.

All of these standards serve to broaden acceptance and increase confidence in these products. They also serve to establish minimum performance and manufacturing standards for these products.



FIG. 4—From FEMA P-675, installation of profile wall corrugated pipe for a drainpipe replacement during a modification of an embankment dam [25]. Corrugated polyethylene pipe has been used successfully to reline failing pipe by a number of agencies. In 1983-84, the Oklahoma Department of Transportation relined over 40 corroded steel pipe crossdrains. In 2007, the Delaware Department of Transportation relined a failing reinforced concrete pipe under a major thoroughfare in Wilmington, DE, with no disruption of traffic and an estimated savings of nearly \$1 000 000 versus removing and replacing the existing pipe.

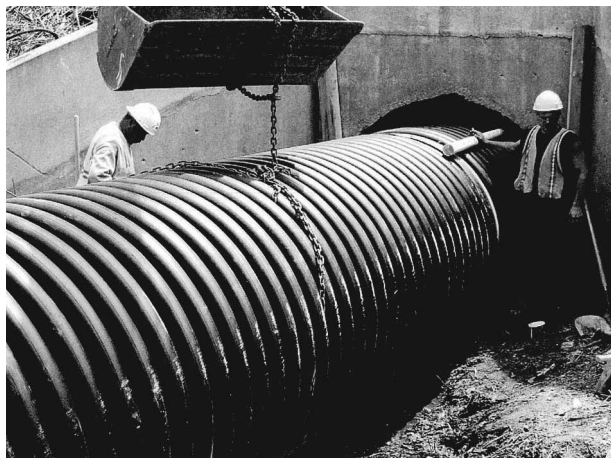


FIG. 5—*Delaware Department of Transportation reline project in Wilmington, DE.*

Currently, all of the state Departments of Transportation specify and use corrugated polyethylene pipe for some applications. Most specify it for culvert and storm sewer applications. The Federal Highway Administration, the Federal Aviation Administration, the Army Corps of Engineers, and other agencies include it in their specifications [23,24]. The Federal Emergency Management Agency, in FEMA P-675, "Technical Manual: Plastic Pipe Used in Embankment Dams," includes guidance on the application and installation of corrugated polyethylene pipe in dams. FEMA, as the lead agency for the National Dam Safety Program, sponsored development of this document in conjunction with the Association of State Dam Safety Officials, Bureau of Reclamation, Mine Safety and Health Administration, Natural Resources Conservation Service, and U.S. Army Corps of Engineers (Fig. 4 and 5).

Summary

The corrugated polyethylene pipe industry in North America today consists of about 20 producers, ranging from large international companies to small single plant operations serving local markets. It has grown from nothing in 1966 to a multi-billion dollar industry. As an industry, it dominates the underdrain/sub-surface drainage market. It has a double-digit share of the culvert and storm sewer market.

What does the future hold? There will be continued growth in markets served and market share of existing markets for the foreseeable future. Additional companies or producers will probably enter the market over the next few years, especially companies currently producing other pipe types. Currently, there are 6 manufacturers of corrugated polyethylene pipe in North America whose principal business is manufacturing corrugated metal pipe. There is one company producing corrugated polyethylene pipe whose principle business producing concrete pipe. There is one major producer of PVC pipe also manufacturing and marketing

corrugated polyethylene pipe. It is reasonable to assume this trend will continue. It is also reasonable to expect that larger diameters will be produced, though probably limited to 96 in. (2400 mm) for the foreseeable future. Specifications will continue to change as the industry changes.

Certainly, one of the driving factors in this growth is and will be the demand for increased competition. Competition is a significant factor in reducing costs. As noted by Adam Smith in "Wealth of Nations" published in 1776, competition reduces costs and improves products.

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The Hydrostatic Stress Board of Plastics Pipe Institute: The First 50 Years

ABSTRACT: The activities and the policies issued by the Hydrostatic Stress Board (HSB) of the Plastics Pipe Institute have significantly enhanced the reliability of pressure rated thermoplastics pipe and the quality of ASTM, American Water Works Association, and other standards that cover such pipes. HSB policies define procedures for the forecasting of the long-term hydrostatic strength of thermoplastics piping materials and for the reducing of this strength into a hydrostatic design stress (HDS). HSB policies have been developed and are periodically updated under the light of our most current understanding of the factors that affect significant strength under actual service conditions. In support of the development of the most appropriate policies the HSB has also conducted various investigative activities. This paper reports on the major contributions that have been made by the HSB. Furthermore, it does so with reference to an overview of the factors that are now recognized as affecting the significant strength under actual service conditions and those that need to be considered when reducing a particular measure of strength into the most appropriate value of a HDS for an intended application.

KEYWORDS: thermoplastics pressure pipe, hydrostatic design stress, long-term hydrostatic strength, design factor, service factor, fracture mechanics behavior, standards

Introduction

Fifty years ago thermoplastic pressure pipe was a specialty material only used in a limited number of applications. Today, it is very widely accepted for the distribution of water and gas, for industrial uses, for plumbing, and for

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rehabilitation of older pipes. This acceptance, still growing, was made possible by the emergence of thermoplastic pressure pipe as an engineering material, a material offering predictable and reliable performance. A very important contributor to this emergence has been ASTM Committee F17, Plastic Piping. However, an even earlier and still a steady contributor is the Hydrostatic Stress Board (HSB) of the Plastics Pipe Institute (PPI). This brief report presents a history of the HSB and a summary of its past and continuing contribution to the ensuring that notwithstanding its point of manufacture, thermoplastic pressure pipe is of high quality and very reliable. This report also offers an opinion of the current state of the art of the forecasting of a material's working strength for an intended application and, it suggests a direction deserving further attention.

This summary and opinions are those of a person who in the earliest days of his career was privileged to be invited to be one of the first members of the HSB. Twenty years later, when Dr. Frank Reinhart retired from PPI, this writer was invited to join that organization and replace Dr. Reinhart as the Chairman of the HSB. Fifteen years later, when this writer retired from PPI as its Executive Director he was invited to stay on the HSB as a member, a position he continues to hold through this day.

This report confines itself to the principal issue of forecasting a working stress for the conveying of water. The long-term strength of thermoplastics can be affected by certain substances. If so affected, but the material still offers serviceable strength, the convention is to reduce the working stress for water by means of an add-on environmental factor. Such factors are presented in engineering information that is available from the various thermoplastic pipe associations.

A Bit of Early History

Responding to a Major Challenge for an Emerging Industry

The Working Stress Subcommittee (WSS), the predecessor to what is now the HSB was formed in 1958 by the Test Methods Committee (TMC) of the Thermoplastic Pipe Division (TPD) of the Society of the Plastics Industry (SPI). In 1963 the TPD changed its name to the PPI. Furthermore, in 1999 PPI became an independent organization.

The WSS was formed for the achieving of two objectives: First, to develop a method for the forecasting of the long-term hydrostatic strength (LTHS) of thermoplastic pipe and, second; recommend a design (service) factor by which this strength should be reduced so as to establish a hydrostatic design stress (HDS) for the transport of water.

This subgroup was formed because of the chaos that then existed in the determination of the HDS for a thermoplastics pipe. Some determined it based on a pipe's short-term burst strength; some used longer-term strength burst data that were obtained by a variety of non-standard methods; some based design on projections of long-term creep; and some based it mostly on observed quality of experience. Also, different equations were then used for computing a pipe's resultant hoop stress.

The initial nine members of the WSS were carefully selected on the basis of (a) technical competence, (b) willingness to allot significant time for this work, as well as for any needed laboratory work, (c) willingness to represent the industry as a whole rather than just a narrow interest, and (d) personal integrity. These same membership requirements persist through this day. Most, but not all, of the laboratory and other studies that led to the development of the earliest version of the method for the forecasting of longer-term hydrostatic strength resulted from contributions made by this group. Before describing the development of the method, first a short description will be provided about other important contributions that have been made by the early PPI towards the creation of today's standardization system for thermoplastics pipe.

Laying the Foundation for a New Industry

The Thermoplastics Pipe Division of the SPI, the precursor to what is now PPI, was created in 1951. The thermoplastic pipe industry in that year was then still in its infancy but, the rapidly growing acceptance of thermoplastics pipe promised a bright future. The early leaders of this industry recognized that the extent to which this promise is achieved depends on actions that work to ensure that thermoplastics pipe becomes recognized as safe, reliable, and very long lasting. That is, the transformation of thermoplastics pipe from a specialty into a widely recognized engineering material.

One of the earliest actions of PPI was the formation of the TMC, the parent of the Working Stress Subcommittee. However, this trade association has also contributed in other important ways towards the development of the thermoplastics pipe industry, including the following:

- (1) A major first action was to retain the National Sanitation Foundation (now NSF International) for the conducting of an evaluation of the fitness of thermoplastics pipe for the conveying of potable water. This comprehensive study, completed in 1956, concluded that the thermoplastic resins that were then used were very fit for this application. However, this NSF study also showed that this fitness could be compromised by the introduction into the pipe formulation of improper additives. Based on this, NSF proposed to PPI a program by which an NSF mark that is placed on a pipe attests that based on criteria established by NSF the pipe is approved for the conveyance of potable water. PPI accepted this program and by this action thermoplastics pipe became the first pipe that was required to meet a health based standard. By this action the NSF took the first step towards what turned out to be a major involvement in documenting the engineering fitness of thermoplastics piping for many other applications.
- (2) In 1952 PPI members voted to participate in a voluntary U.S. Department of Commerce program that issued Commercial Standards which were proposed by an industry group and which were judged as technically appropriate following a review by a representative of the National Bureau of Standards, then a unit of the Department of Commerce. The representative assigned to PPI was Dr. Frank Reinhart, chief of the

bureau's Plastics Section. In 1954 the first Commercial Standards were issued for polyethylene (PE), cellulose acetate butyrate (CAB), acrylonitrile-butadiene-styrene (ABS), and polyvinylchloride (PVC) pipes. At first Dr. Reinhart provided very valuable guidance to PPI, but in ensuing years his involvement with this organization vastly increased. In 1962, when the Plastics Section of the Bureau of Standards was abolished Dr. Reinhart agreed to be PPI's Technical Director and to be the permanent chair of its Working Stress Subcommittee. In later years, as ASTM standards gained preferential acceptance, Commercial Standards for thermoplastic pipe were withdrawn.

- (3) In 1953 PPI retained Batelle Memorial Laboratories for the identifying of the most technically appropriate test methods that should be used in evaluating thermoplastic piping materials and the pipes made from such materials. In 1955 this program was expanded so as to allow Batelle to construct recommended test equipment and use it for the testing of pipes. A second objective was for Batelle to develop a recommended methodology based on which the long-term strength of a thermoplastic piping material may be forecasted. The first of these objectives led to the development of test methods which later became recognized under the following standards:
 - (a) ASTM D1598, "Time-to-Failure of Plastic Pipe Under Constant Internal Pressure" [1].
 - (b) ASTM D1599, "Short-Time Hydraulic Failure Pressure of Plastic Pipe, Tubing and Fittings" [2].

The second of these objectives led to a recommendation by Batelle that the long-term strength of thermoplastic piping materials may be forecasted based on the extrapolation of shorter-time stress-rupture data. To develop the most appropriate forecasting methodology Batelle recommended that extensive stress-rupture testing should be conducted on the then currently available thermoplastic pipe materials. It was largely based on these two recommendations that in 1958 PPI established the WSS, which in a later year was renamed the HSB.

- (4) In 1954, under the recommendation of Dr. Reinhart who was then still with the Bureau of Standards and an advisor to PPI, a joint ASTM-SPI Subcommittee was established under ASTM Committee D20, Plastics. The initial charge of this group, identified as Subcommittee 17, was to develop specifications for thermoplastic piping materials and for test methods. There was a need for such documents by pipe standards that were issued by the Department of Commerce and other bodies. Dr. Reinhart agreed to be the Secretary for this group and PPI provided the necessary secretarial service. In a short time Dr. Reinhart became the most influential technical leader of this ASTM subgroup.

In 1960 the scope of Subcommittee 17 was expanded so as to allow it to develop standards for pipe. This action recognized that consensus standards issued by ASTM are much more likely to be referenced by code bodies than Commercial Standards. In 1973 Subcommittee 17 was elevated to full committee status, Committee F17, Plastics Pipe. This action recognized the large and

growing membership of this group as well as the rapidly increasing number of standards that were being written. Today, in North America, Committee F17 is the far away leader in the development of standards for thermoplastics pipe. The current crop consists of over 110 standards that cover pipe, tubing, liners, fittings, test methods, installation methods, joining, and other practices and terminology.

The emergence of Committee F17 as the major issuer of North American standards for thermoplastic pipe resulted from the work and dedication by many persons, and also by the early secretarial assistance that was provided by PPI. However, two persons deserve most of the credit, Dr. Frank Reinhart and Henry Kuhlmann. During the early years of this ASTM group Mr. Kuhlmann was a leader in various research projects on plastic pipe that Batelle Laboratories was conducting for the American Gas Association. The outstanding contributions made by these two individuals towards the creation of standards for thermoplastics pipe were formally recognized in 1993 by the creation by ASTM of the Reinhart/Kuhlmann award, an award that is now granted to an F17 member in acknowledgement of superior leadership in the development of standards.

- (1) In 1961 PPI adopted certain basic principles, as follows, for the guiding of the writing of standards for thermoplastic pipe:
 - (a) A uniform methodology needs to be used for the determining of the HDS for thermoplastic pressure pipe.
 - (b) Because long-term strength can vary from one formulation to another, even among materials made from the same base resin, each formulation must be evaluated for long-term strength by the generally accepted uniform methodology.
 - (c) The "thin wall" formula—now also known as the International Organization for Standardization (ISO) formula—should be used to describe the relationship between pressure, pipe dimensions and nominal value of hoop stress.
 - (d) The strength reduction factor based on which an HDS is established for an intended service shall be identified as either the service factor or the design factor, but not as the factor of safety.
 - (e) The essential parameters based on which a pipe material and a pipe are defined by a standard—long-term strength, design factor, hydrostatic design stress, ratio of diameter to wall thickness, and pressure rating—shall preferentially be expressed in terms of a preferred number. For this purpose the TPD recommended the numbers in the 10-series of standard ASA ZS17.1-1958 [3]. In this series the next larger number is approximately 25 % above the previous number.
 - (f) A preference for establishing standard diameters based on iron pipe sizes (IPS) and copper tubing sizes (CTS).

The Working Stress Subcommittee's Response to the Challenge

The first meeting of the WSS was held in December 1958. Over the next 30 months a meeting was held every month in 2 or 3 day sessions. At each of these

meetings new data and concepts were reviewed, and the results often led to a revision of an emerging method for the forecasting of the LTHS of thermoplastic piping materials. An early decision of this group was that the method must produce a forecast based on a mathematical procedure. At that time, researchers in Europe had developed a graphic method for PE pipe, the attraction of which was that it utilized relatively short-term stress-rupture data obtained at elevated temperatures for the forecasting of a material's much longer stress-rupture behavior at ambient temperatures (Fig. 1). But, the possibility of using this method was abandoned after conducting an exercise in which Subcommittee members were provided with log stress versus log time-to-failure data obtained at four temperatures on a PE piping material and were asked to produce a forecast of the material's LTHS at 23°C and at the 50-year intercept. The forecast made by each subcommittee member was different and, the range of predicted values varied considerably.

Based on this experience the hope of coming up with a method that only required shorter term stress-rupture data was abandoned. Following the analyses of existing data, it was instead decided that for the producing of a more accurate prediction of LTHS the stress-rupture data should cover a testing period of not less than 10 000 h. Two considerations led to this decision: (1) Should a down-turn occur in the log stress versus log time-to-fail plot prior to the 10 000 h intercept this will result in a lower forecast of LTHS which makes the material uncompetitive when compared to a similar material that does not down-turn, and (2) if a down-turn occurs after 10 000 h a smaller decrease in LTHS will occur, a decrease that it was believed may be compensated by means of the strength reduction factor (the design factor). Figure 1 depicts down-turns that occurred in a very early PE, a material withdrawn soon after its introduction.

By early 1960 over 350 stress-rupture data sets were analyzed by this group. A study of all these data sets, which included sets that showed a down-turn prior to 10 000 h, led to a 9th working draft of a proposed method. This draft was then submitted for review and comment to the parent group, the TMC. The comments received led to a 10th draft, a draft that was submitted to letter balloting by the whole membership of PPI. This draft was approved in 1961, but with some comments. Acceptance of these comments resulted in an 11th draft which became the first "tentative" uniform method for the forecasting of the long-term strength of a thermoplastics piping material.

Six years later, in 1967, some minor revisions were made in this method and its tentative status was dropped. This version was issued by PPI in report TR-2 (1967), "Recommended Method for Obtaining Hydrostatic Design Basis for Thermoplastic Pipe." A year later, after some minor changes, this method was issued by ASTM as standard ASTM D2837 [4], "Obtaining Hydrostatic Design Basis for Thermoplastic Pipe Materials." The hydrostatic design basis (HDB) in the title refers to the reduction by this method of the LTHS that is forecasted into one of a limited list of standard hydrostatic strength values which have been established based on standard preferred numbers. Upon the issuance of D2837 PPI deleted this TR-2 report from its publication list.

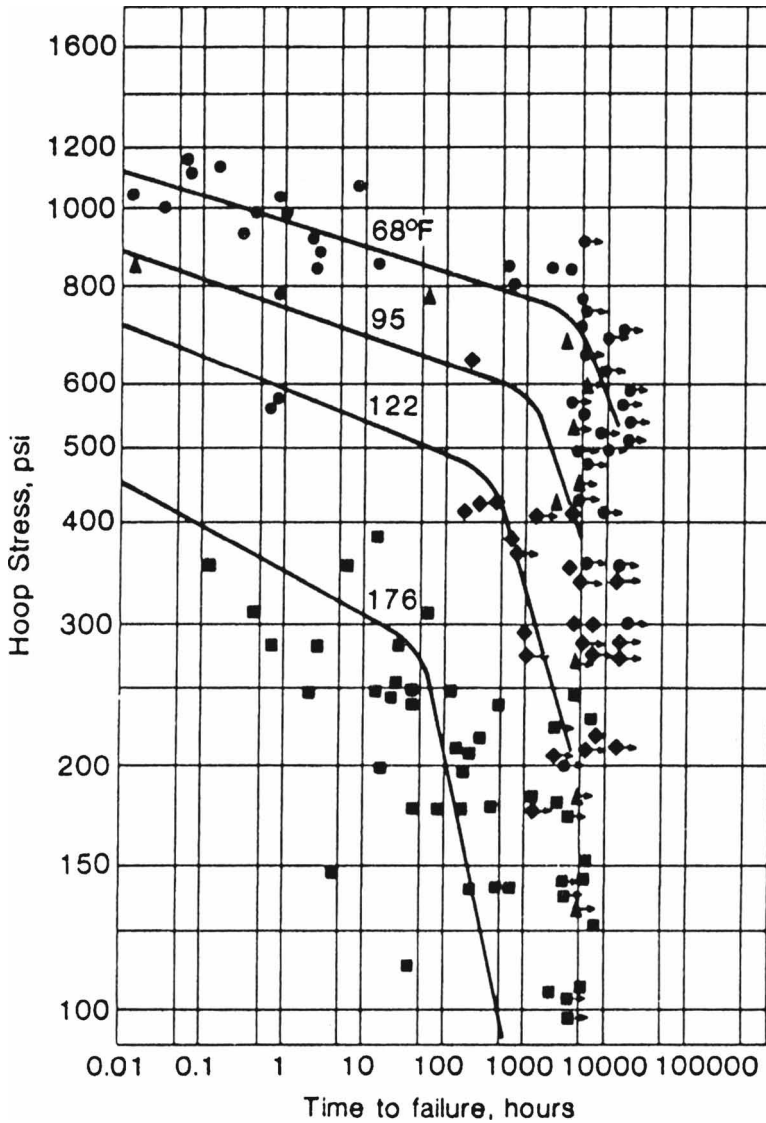


FIG. 1—Effect of temperature (from Richard K. Diedrich and Gaube E., *Testing of Ziegler Polyethylene Pipe*, *Rubber & Plastics Age*, Vol. 39, May 1958, pp. 364–368) on the stress-rupture properties of a certain high density PE pipe material (this material is not commercial in the United States). Arrows indicate test in progress.

The more significant features of the PPI method that have been carried into this first and in future issues of ASTM D2837 are as follows:

1. The method is only applicable to materials for which their log stress versus log time-to-fail behavior is expected to follow an essentially straight-line course.

2. The method assumes that if during the first 10 000 h of testing the resultant data fall on a straight line then this is a sufficient indication that this same behavior shall continue through at least the 100 000 h intercept, which marks the point at which the long-term strength is forecast. The method includes a statistically based procedure to determine compliance by the obtained data to the method's requirement of a straight-line behavior over the first 10 000 h.
3. A material's LTHS is its projected mean value of hydrostatic strength at the 100 000 h intercept. Based on statistical considerations it was decided to limit the extrapolation to 100 000 h, one log cycle beyond the required minimum 10 000 h test period. During the study of available data sets it was noted that the analysis of different data sets for the same material indicated a lesser variance in the predicted mean than in the predicted lower confidence limit (LCL). This difference was attributed not only to variations in different lots of the material but mostly to variabilities in the testing procedure. Because of this, it was decided that the value of the LTHS should be based on the predicted value of the mean and that the expectation of a minimum predicted value shall be compensated by means of the service factor.
4. The LTHS that is forecast is then reduced to one of a list of preferred values which is designated as the hydrostatic design basis (HDB).
5. To establish a HDS the document advises that the HDB should be reduced by a service or a design factor that gives consideration to all the variables that can affect the in-service long-term strength, and that this factor should also compensate for the expectation that the service life (the design life) of the pipe life is intended to be much longer than the 100 000 h intercept at which the HDB is determined.

Regarding the last item, a recent paper [5] states that the time intercept at which the LTHS is determined for a particular material is the determinant of the design life of the pipe that is made from that material. This notion is misleading for it ignores that the design life for a certain application is established in consideration of two essential factors: (1) The nature of the method that is used to determine strength, and (2) the compensation (the service or design factor) that should be applied to this measure of strength so as to give adequate consideration to all the in-service conditions and expected duration of service which are not considered by the method based on which LTHS is determined.

During the development of this method there were held many discussions regarding the magnitude of the strength reduction factor that should be applied to the HDB so as to arrive at the most appropriate HDS for pipes intended for the transport of water. An early decision was that this reduction factor should not be labeled a *factor of safety* because such label is misleading. It was decided to designate it as the *service factor*, or the *design factor* because in addition to the providing of a safety margin the other major functions of this factor is to compensate for various other realities, including the following: The conditions under which an HDB is determined do not replicate all the conditions that a pipe sees in actual service (under the test procedure the pipe is subjected to a sustained stress and a sustained temperature whereas under service conditions

the pipe pressure rating and the temperature represent maximum operating values); the HDB is determined based on a projected mean strength at the 100 000 h intercept (about 11 years) and not based on a projected minimum strength over the pipe's expected service life; there are normal variabilities in the manufacture of the material and pipe which can affect strength; and, there are also variabilities in the handling, in fabrication (pipe and fitting joining procedures) and, in quality of installation that can affect strength and resultant stress. Based on these considerations and also on the results of actual field experience the initial suggestion from PPI members was to reduce the HDB by means of a divisor that ranged from 1.5 to 2.0. The WSS favored the 2.0 divisor. In 1960 this question was referred to the whole of the membership of PPI and they voted by a narrow margin to adopt the 2.0 factor for all materials on a trial basis. The next popular choice was 1.8. After a trial of 2 years the PPI membership officially adopted the 2.0 factor for all materials. In 1968 it was decided by PPI that for the minimizing of the too frequent tendency to wrongly view a *service factor* or, *design factor* as just being another name for *factor of safety* it was decided to express this factor as a multiplier (a number less than 1.0). Soon after, this convention was accepted by all standards issued by ASTM as well as by most other organizations. This is how the initially accepted 2.0 service factor for water, a divisor, became a 0.50 multiplier. From its initial issue ASTM standard D2837 has recognized the terms service factor and design factor. ASTM F412, "Standard Terminology Relating to Plastic Piping Systems," [6] defines service factor as follows: "a factor that is used to reduce a strength value to obtain an engineering design stress. The factor may vary depending on the service conditions, the hazard, the length of service desired, and the properties of the pipe."

The Working Stress Subcommittee Becomes Permanent Group

In early 1963, 2 years after PPI's acceptance of the Working Stress Subcommittee's recommended method for determining the HDS for thermoplastic pipe materials for water, PPI's membership requested that the WSS establish a program by which it shall issue public recommendations of HDS for water, for 73.4°F for pipe thermoplastic pipe materials. In mid-1963 it issued its first HDS recommendations. These were for pipe materials made from PE, PVC, CPVC, and ABS resins. In recognition of its new and permanent role in 1967 this group was renamed the Working Stress Committee. Then in 1983 it was again renamed, this time to the HSB, its current name.

The initial listing of only a few recommendations for HDS has grown to a major program based on which the HSB now also issues recommendations that are referenced by standards for determining the pressure rating of a pipe or fitting. The policies for PPI listings are identified in PPI report TR-3, "Policies and Procedures for Developing Hydrostatic Design Basis (HDB), Hydrostatic Design Stresses (HDS), Pressure Design Basis (PDB), Strength Design Basis (SDB) and Minimum Required Strength (MRS) Ratings for Thermoplastic Pipe Materials or Pipe." A listing of HSB recommendations is offered in the periodically updated report PPI TR-4, "HDB/HDS/SDB/PDB/MRS Listed Materials." The current issue of TR-4 lists recommendations that are close to 1 000 in number, a

total that gives impressive testimony of the widespread acceptance of this program.

In 1963 was also initiated a new program by the NSF which continues through this day. The intent of this program is to give assurance to a purchaser of thermoplastic pipe that when the pipe carries a special NSF logo this designates that the pipe has been determined by NSF to comply with all the performance requirements of the standard to which it has been made, including its manufacture from a PPI listed material. In regards to this latter requirement, NSF also requires that when a PPI listed formulation is modified the modification can only be made in accordance with the policies in PPI TR-3. Because of this involvement NSF has a permanent representative on the HSB. This collaborative effort by PPI and NSF is recognized as the most important factor in the making of thermoplastic piping into a high quality and very reliable product.

The initiation of the development of the policies that are now listed in PPI report TR-3 was also in response to an early request from the PPI membership. When the industry first accepted the new uniform method for establishing HDS's it was not long before it saw the lengthy time that it takes to develop data as a major challenge in obtaining a listing for a new material and for qualifying a change in an already listed formulation. In 1967 the PPI membership asked for some relief of this situation. After a few months the WSS responded as follows:

- In the case of a member of a family of piping materials for which previous testing indicates that this family follows the straight-line behavior that is assumed by the extrapolation method, a series of temporary Experimental Grade Recommendations will be granted to that member so as to allow sufficient time for the development of the full data requirements of the extrapolation method. The first temporary Experimental listing requires that the data span at least 2 000 test hours, the second 4 000 h, the third 6 000 h and so on, until the full requirements of the Standard Grade are met. Since the inception of this system it has been a very rare instance that a material which has been assigned an Experimental listing fails to make the Standard Grade.
- The WSS will establish policies that will simplify data requirements based on the nature of the proposed change in a formulation. If a substitute ingredient is identical to the original based on chemical and physical requirements established by the WSS no data for the qualifying of the substitution will be required. However, if the substitute ingredient cannot be thus defined a policy will be established which will call for simpler data requirements in cases where a simplification is justified based on what has been learned regarding the effect of that change on the long-term strength of the pipe material. In addition, the WSS will establish a Special Case opportunity for hearing proposals for simpler data requirements for cases not covered by existing policies.

Through the years the second objective has received a great deal of attention from the HSB. This led to many new helpful policies which have greatly facilitated the qualifying of alternate ingredients, while ensuring that the stress-rupture behavior demonstrated by the original formulation is not compromised. These policies are presented in the aforementioned TR-3 report.

In the early 1970s a new trend was beginning to take place in the production of PVC pipe. Instead of extruding pipe from pellets that have been pre-compounded by the resin supplier, PVC pipe extruders were increasingly making pipe from a dry blend that was prepared at the extruder's facility. This new practice was recognized by the institution of a *dependent* listing. This kind of listing is granted to an extruder under two conditions: (1) The extruder has to affirm that the subject formulation to which he has been given access—an *independent* formulation which is owned by another party—shall be prepared in accordance with the procedure that is defined by its owner, and (2) the owner of the formulation must inform the HSB that permission has been granted to the extruder for the use of the subject formulation.

At the beginning of the 1980s dependent listings were also started to be issued to extruders of PE pipe.

In 1980, acting on a request from various PVC pipe extruders, a generic and PPI owned PVC range formulation was established. The development of such a composition was made possible because of the consistency of the chemical architecture of the PVC resins that were used in pipe production, even though the resins were made by different manufacturers. Currently, over 50 extruders of PVC pipe are listed as users of this composition. This generic composition permits the use of alternate PVC resins and ingredients, but each must have been previously qualified by longer-term stress-rupture testing. PVC resin suppliers and pipe extruders cooperated in the development of this formulation by releasing what at one time was restricted information and data. The details of this formulation are presented in PPI report TR-2, "PPI PVC Range Composition, Listing of Qualified Ingredients."

During the late 1970s there began to emerge occasional reports of small brittle-like failures in PE pipes that occurred not because of the pipe's inadequate resistance to internal pressure—the significant loading which thermoplastic pipe is designed to safely resist—but because of the effect of a localized stress intensification caused by stone impingement, or by differential soil settlement, or by sudden change in geometry from pipe to fitting, or by improperly made fusion joints, etc. These events appeared to be linked to PE resins that in 73°F stress-rupture testing were believed to exhibit a down-turning after 10 000 h but prior to 100 000 h. As such reports continued to emerge, in 1983 the HSB decided to conduct a special study regarding this matter. By that year an analytical tool became available based on which the time to rupture in the down-turn region in the 73°F stress-rupture behavior of a PE pipe may be predicted by the analysis of the rate at which the time to failure at the same stress is accelerated by testing at elevated test temperatures. Using this tool, which was developed on rate process principles that were found to also apply to the brittle-like failure mechanism of thermoplastics (the rate at which a crack is formed and then slowly grows is governed by both the driving force—stress, in this instance—and absolute temperature), this study determined that such very localized failures were only taking place in PE's for which this tool predicted that a down-turn (the indication of a shift from ductile to brittle mode of failure) will occur in the 73°F stress-rupture characteristics at some time prior to the 100 000 h intercept. This HSB study showed that the sooner the down-turn was predicted

to occur the more vulnerable was the PE pipe to the effect of localized stress intensifications. There were three important lessons derived from this study:

- (1) It challenged the essential assumption upon which the design of thermoplastic pipe is based. Namely, that the significant strength is the material's capacity for safely resisting the hoop-stress that is only induced by hydrostatic pressure. Most of the failures occurred at low operating pressures.
- (2) It also challenged the validity of the assumption by method ASTM D2837 that the existence of a straight-line log stress versus log time-to-fail relationship through the first 10 000 h of testing gives adequate assurance that this same straight-line course shall be followed through at least the 100 000 h intercept, the point at which the LTHS is determined. It showed that compliance to this key assumption of D2837 needs to be validated by other means.
- (3) The down-turn in stress-rupture properties results from a shift from a ductile to a brittle-like mechanism (the development and slow growth of cracks) that causes strength and strain at failure to decrease more rapidly than indicated prior to the shift. This study showed that the brittle-like failure mechanism makes thermoplastics more susceptible to failure by the effect of a localized intensified stress. Therefore, the occurrence of a down-turn in stress-rupture properties is an indicator of this susceptibility.

In response to these lessons, and by the application of rate process principles, this group developed an elevated temperature test procedure for PE pipe materials for the validating of the assumption behind method ASTM D2837, namely that a straight-line behavior through 10 000 h of testing is sufficient evidence for the assuming that this behavior shall essentially continue through at least the 100 000 h intercept. This validation method for PE was adopted by PPI in 1985 and, in 1988 it was incorporated into ASTM D2837.

Nearly 25 years of experience after the implementation of this validation requirement shows that PE pipe's vulnerability to localized stress intensifications has been eliminated. A further benefit is that the availability of this methodology has given major guidance for the changing of the chemical architecture of the newer PE pipe resins. Current PE resins do not exhibit a down-turn at 73°F, and even at higher temperatures, until significantly past the 100 000 h intercept. This has led to the institution in 2001 of a new requirement, called substantiation. A stress-rupture line is substantiated when it is demonstrated by means of supplementary elevated temperature testing that a down-turn at 73°F is not predicted to occur until past the 50-year intercept.

A difficulty of the current requirements for validating and substantiating is that the required testing takes a very long time to complete—in some cases, longer than a year. Based on information that indicates that cyclic stressing greatly shortens the time to failure by the slow crack growth process the HSB has retained a laboratory for studying this possibility. Results of this study have given a strong indication that a correlation between the two failure times is achievable [7]. As a result, a further evaluation is in the planning stage.

Another important factor guiding the development of the newer PE resin has been the issuance by ASTM in 1997 of test method F1473, "Standard Test Method for Notch Tensile Test to Measure the Resistance to Slow Crack Growth of Polyethylene Pipe and Resins" [8]. Under this fracture mechanics based test method a sharply notched sample is subjected to a specified tensile stress at an elevated temperature. The time to-failure under this test gives an index of a material's resistance to slow crack growth under the effect of an intensified stress. By means of studies conducted on older PE pipes that have been found to be vulnerable to brittle-like failure under installation conditions that produce localized stress intensifications, the failure time under this method has been determined to provide an additional and very useful index of a PE pipe's resistance to the formation and slow growth of cracks.

In the U.K., certain PVC pipes were also found to fail prematurely at points of stress intensification caused by point loading but not because of hydrostatic pressure. The remedy to this problem was the development of a fracture toughness test the objective of which is to ensure high resistance to point loading [9].

As it has been determined for metals, localized stress intensifications also occur in plastics which are under stress. These intensifications can result from what may be termed as internal or external causes. An example of the former is irregularities in the material caused by voids, gel particles, additives, etc. These irregularities create localized stress risers that can lead to the formation and slow growth of cracks. For this reason HSB has issued policies that control the quantity and the nature of additives that are added to a thermoplastics pipe formulation.

An example of an external cause is the point loading in a buried pipe that occurs due to rock impingement. Localized intensified stresses can also occur at pipe joints, at locations subject to differential settlement and at locations of sudden geometrical changes. Experience shows that in thermoplastics piping it is the external causes that result in premature failures. Although all intensified stresses can be diminished by quality of piping material formulation, and by careful handling, fabrication and installation of pipe, and also by reducing the pipe design stress, the best solution is to demand from plastic piping materials a very high tolerance to the effect of localized stresses. Thus, a pipe material must not only exhibit adequate hydrostatic strength—a parameter that only evaluates the effect on strength by hydrostatic pressure and one that ignores the information conveyed by the mechanism of failure—but also, adequate resistance to the localized stress intensifications that are expected to be seen in actual service. For this reason have emerged various new supplementary tests intended to ensure a higher quality of resistance to localized intensified stresses.

In 2000, the HSB Decided to Include in Its TR-4 Report Recommendations of Hydrostatic Strength That Have Been Determined in Accordance with Standard ISO 9080

As a service to the industry, the HSB thought it appropriate to also list LTHSs which have been determined in accordance with the requirements of this ISO standard. The principal difference between this standard and ASTM D2837 is

that the ISO standard requires that stress-rupture data be also collected at temperatures above the design temperature and that it employs a more complex mathematical tool that allows it to forecast a LTHS whether this strength lies in the region before or after the occurrence of a down-turn. Based on the forecasted minimum value of this strength, for water for 20°C (68°F), a material is then assigned to one of a limited number of standard hydrostatic strength categories labeled minimum required strength (MRS). While the capability of ISO 9080 to forecast a hydrostatic strength that lies beyond the point at which down-turning occurs is considered by some to be a great plus, there is growing recognition that this down-turning region is caused by the growth of cracks and that it also represents a region in which failure occurs by a slow crack growth process and where strength and strain at failure regress much more rapidly under the effect of increased duration of loading. A lower strain capacity is recognized as an index of a lesser tolerance for the safe absorption of localized stress intensifications. Thus, an MRS rating that represents a strength that lies in the down-turn region is not the equivalent of an MRS rating that represents a strength that lies prior to this region – the ductile failure region. In Australia, PE materials that achieve their strength rating in the down-turn region are not favored for water distribution applications [10]. The user of MRS listings also needs to be aware that most European and other thermoplastic pipe standards which reference this ISO method also include supplemental pipe material requirements that are designed to ensure adequate resistance to localized stress intensifications. The effect of these supplemental requirements is to greatly increase the duration of loading at which there occurs a down-turning in the material's stress-rupture behavior at ambient temperatures. For materials that satisfy these supplemental strength requirements ASTM D2837 and ISO 9080 forecast relatively equivalent values.

The HSB Recommends a Higher Service Factor of 0.63 for High Performance PEs

Nearly 50 years of actual experience has shown that the HDS that is established by the reducing of an HDB by a 0.50 design factor (DF) has been a very successful practice—seldom, if ever does a thermoplastics pipe fail because of inadequate resistance to hydrostatic pressure. However, experience and studies show that when failure occurs it results from inadequate resistance to localized stress intensifications and not in consequence of internal pressure. In other words, what this experience shows is that the design system we established is very successful in ductile behaving materials in which the significant stress is that which results from the effect of hydrostatic pressure; but it is not as successful in brittle behaving materials for which the significant stress is that which is created by localized stress intensifications. What this teaches is that a thermoplastics pipe, in addition to offering adequate resistance to the stress generated solely by hydrostatic pressure, must also offer adequate resistance to resultant localized stress intensifications. It also demonstrates that a major function of the design factor (DF) is to adequately compensate for the nature of a material's reaction to the larger stresses that are generated by localized stress intensifications. The DF does not need to be as large for materials that greatly

resist the effect of localized stress intensifications as that for materials that display lesser resistance.

During the past two decades have been developed very ductile behaving PE's that exhibit outstanding resistance to localized stress intensifications. This resistance is evidenced through various tests including sustained pressure tests in which the pipe was simultaneously subjected to internal pressure and a locally applied stress [11], as well as by certain other fracture mechanics based tests. In stress-rupture testing these newer PE's materials fail by a yielding mechanism, even under very long periods of loading which tends to favor the development of the brittle-like failure mechanism. In the case of materials for which long-term strength is delimited by ductile yielding and not by brittle failure prior to yielding it is recognized that their design stress can be set higher than for materials that are susceptible to brittle-like failure. This is the basis for the assigning of a DF of 0.63 for these newer PEs.

For the differentiating of these higher performance PEs from the traditional PEs the following criteria have been established by the HSB:

- Supplemental elevated temperature testing must indicate that an initially established straight straight-line behavior for 73°F between log stress and log time-to-fail, a consequence of a ductile failure mechanism, shall continue through at least the 50 year intercept.
- The ratio of the predicted LTHS to its LCL must not be less than 0.90.
- At the prescribed elevated test temperature of 80°C, the material must exhibit a minimum resistance to slow crack growth of 500 h when tested in accordance with ASTM method F1473.

On Establishing the Most Appropriate Hydrostatic Design Stress for Thermoplastic Piping Materials

To avoid failure the maximum stress (the significant stress) acting on some point in a pipe must be less than the strength (the significant strength) that resists this stress. However, for the design to be practical the significant stress cannot be too much below the significant strength. There must be a proper balance between the two. The ratio between significant strength and significant stress is the true factor of safety. In the relatively few cases in which both the significant strength and significant stress can be accurately predicted, engineering textbooks state that the value of the stress (the design stress) can safely be set as high as 80 % of the significant strength (equivalent to a 1.25 true factor of safety). However, because predictions of significant strength and significant stress are difficult to make, the common practice is to conduct design based on a strength that is readily determinable by means of some standard test method and also based on a major resultant normal (i.e., average value) stress that is also more easily determined. The ratio between these two represents a strength reduction factor, which in addition to the providing of a safety margin, must adequately compensate for the limitations that are imposed by the employ of these two tactics.

Unfortunately, a frequent practice is to also call this strength reduction factor as a *factor of safety*, even though the achieving of a margin of safety is but one of

its principal roles. This nominal factor of safety is larger for brittle than ductile behaving materials because the design stress must be reduced to a greater extent so as to provide a sufficient cushion for the safe absorption of localized stress intensifications that are not directly considered by the design practice. In the case of ductile behaving materials the determining stress is not a localized intensified stress but the average value of the imposed stress. This is because ductile behaving materials can safely shed-off localized stresses. Thus, for ductile behaving materials the nominal factor of safety can be smaller. Obviously, the magnitude of a nominal factor of safety is not a true indicator of the actual margin of safety. In recognition of this problem many codes and standards specify a material's allowable design stress for a certain application rather than a nominal factor of safety. This is the convention that was chosen by the WSS back in the early 1960s. This is why it issues recommendations of design stress for water, for 73°F, and this is also why the word stress appears in its name.

Because the magnitude of a nominal factor of safety, a divisor, can be misleading in regards to actual margin of safety, engineers have chosen to reduce a laboratory measure of strength by means of a multiplier which is termed as either a *service factor (SF)* or, a *design factor (DF)*. This approach was chosen by the WSS back in the early 1960s. And, these terms are recognized by the ASTM standard on terminology for plastics pipe.

In the case of metals, a lower resistance to brittle failure has from the earliest time been recognized by the assigning to these materials a larger nominal factor of safety. The essential function of this larger nominal factor is to decrease the magnitude of the design stress which in turn decreases the magnitude of localized stress intensifications. This tactic has worked rather well, but not always. Starting in the early 1800s, when stronger steels began to be more frequently used, some structures, including bridges, ships, wheels, storage tanks, pressure vessels, and pipes failed unexpectedly even though they were designed by the applying of large nominal factors of safety. This problem drew the attention of engineers and it led to the establishment in the early 1900s of the discipline of fracture mechanics. Although at first it concentrated on metals this discipline has broadened to cover all kinds of materials, including plastics.

The essential lesson that this discipline teaches is that for the effecting of the most efficient and safe design a structural material needs to exhibit high resistance to failure by the growth of cracks. Towards this objective have emerged certain understandings regarding the behavior of engineering materials, and also of the requirements for their selection for an engineering application. The more important items that are believed to also apply to thermoplastic pipe are next listed.

- (1) The formation and subsequent growth of cracks is an irreversible process. When this process takes place the rates at which cracks form and grow becomes the principal determinant of the useful life of a structure.
- (2) Cracks are initiated by the higher local stresses and strains that occur at irregularities in the material or, at irregularities caused by handling, method of installation or by sudden changes in geometry. Localized

stresses can be considerably greater than the surrounding normal (average) stress.

Figure 2 illustrates the stress magnification that can occur at the tip of a flaw. The extent of magnification depends on the geometry of the flaw, particularly at its tip.

- (3) Once initiated, a crack can grow very slowly until it reaches a size at which a failure results (by a leak, by tearing, by cracking or, by other terminal event).
- (4) A recognized index of a material's capacity for safely resisting crack formation and growth is strain at which it fails in a standard test. The larger this strain the greater is this resistance. For metals, a minimum tensile test strain requirement is 5 %, but usually it is 10 % or larger. However, maximum strain capacity is achieved when a material fails by a yielding (ductile) mechanism. Prior to yielding there is no irreversible damage.

Discussion: A demonstration of a PE's capacity to fail in the stress-rupture test by the ductile mode through the 50-year intercept is a principal requirement for assigning to that PE a design factor of 0.63.

- (5) A failure that results in consequence of the development and growth of a crack is symptomatic of a material's brittle strength, whereas a failure that results by a yielding (deformation) process represents ductile behavior. A brittle fracture occurs at nominal (average) stress and strain that is lower than what would have been if the material would fail by yielding. The lower is the brittle failure stress the lower is a material's capacity for safely absorbing localized stress intensifications. This is why fracture mechanics experts state that in a strength test the nature of the failure (ductile versus brittle) needs to be reported.

Discussion: Neither method ASTM D2837 nor ISO 9080 give recognition to these indicators of fracture mechanics behavior. However, as discussed earlier in this report, method D2837 includes a straight-line stress-rupture requirement that is intended to exclude it from being applied to materials that are predicted to fail in a down-turn region in which, because of a change in failure mode from ductile to brittle, stress and strain at failure decrease more rapidly with increased duration of loading. In contrast, ISO 9080 accepts any forecasted value of strength independent whether the strength lies prior to or, in the down-turn region.

For some materials there have been established supplementary strength requirements, the object of which is to ensure high resistance to the development of cracks. These requirements avoid or, very greatly delay the occurrence of a down-turn. For materials that meet these requirements the ASTM and the ISO methods become relatively equivalent.

- (6) Both ductile and brittle behavior of a material depend on the conditions to which the material is subjected (type of loading, duration of loading, extent of straining, temperature, environment, etc.) as well as on the

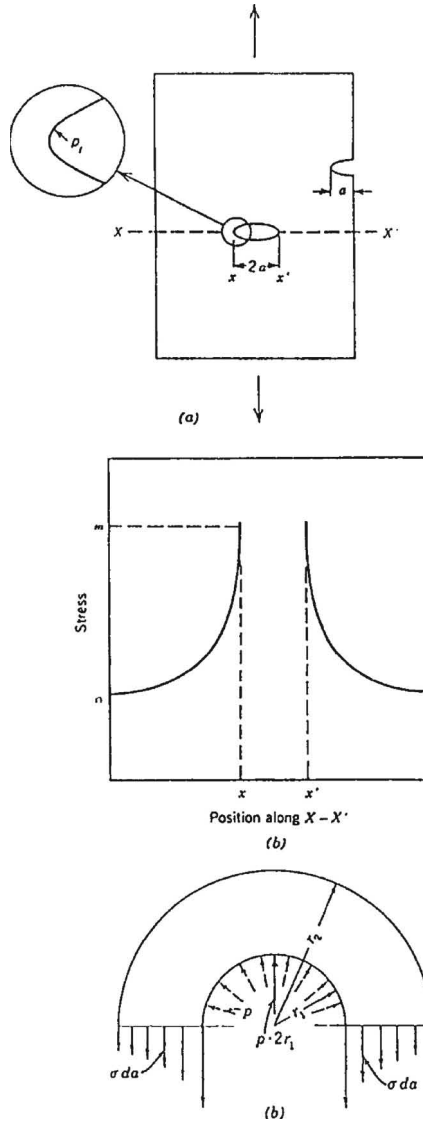


FIG. 2—(a) In brittle behaving materials the geometry of a flaw determines the magnitude of the significant stress; (b) in ductile behaving materials the significant stress is the average value of the fiber stresses.

nature of the material. Hence, strictly speaking, a material should not be designated as ductile or brittle. Instead, a material should be identified for its capacity to operate under actual service conditions in either the ductile or, in the brittle state.

Discussion: As shown by Fig. 3, the boundaries within which a material can operate in the ductile state depend not only on the nature of

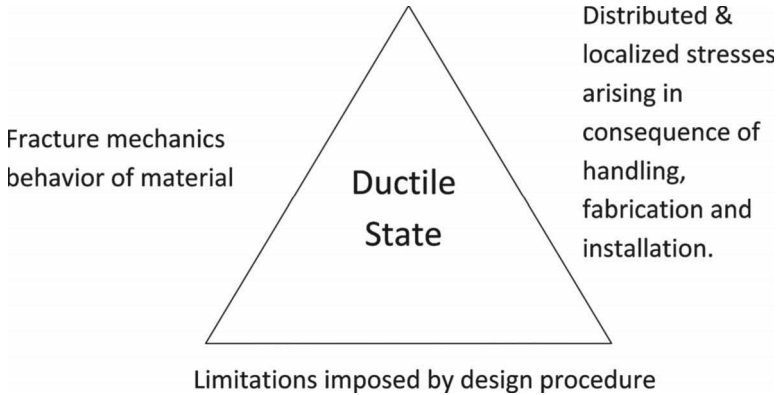


FIG. 3—*Boundaries which define the ductile state.*

the material but also on the duration and the nature of the stresses that act on the material in the intended service.

In essence, what the above items teach is that for the establishing of an appropriate value of design stress to a thermoplastic pipe material it is important to do it by considering more than just the nominal (average) stress at which the material fails in a stress-rupture test that subjects the test pipe to only the effect of internal pressure.

In Conclusion: An Effective Foundation Has Been Established, but There Is More to Be Done

Unquestionably, the broad and still growing acceptance of thermoplastics pressure pipe is in major part due to the availability of standard methods for the forecasting of a material's long-term hydrostatic strength. ASTM Committee F17 deserves much credit for this. It is also due to thermoplastic pipe's very high reliability and durability to which the policies of the HSB have greatly contributed.

But there is yet more to be done. The two methods that are most widely used for the forecasting of LTHS, namely ASTM D2837 and ISO 9080, only concern themselves with effects of internal hydrostatic pressure. And neither of these two methods identify the mechanism of fracture (e.g., ductile or, brittle) that corresponds to a predicted strength. But extensive experience shows that in the infrequent cases of premature failure, the failure results from the effect of non-hydrostatic stresses. As is the case of metals and all other materials, it is the fracture mechanics behavior of thermoplastics that is the ultimate determinant of their long-term structural behavior.

This understanding has already led to the development of additional requirements and of supplementary test methods for the ensuring of adequate resistance to loadings other than those only induced by hydrostatic pressure. To further increase the reliability of thermoplastics pressure pipe more of such requirements and test methods are needed. Of the methods that are already

available certain ones, like substantiation, require a very lengthy evaluation. Ideally, a test method should be capable of qualifying a material for an intended application in as short a time as is possible. The double objective of high significance of test results and short time of testing leave much opportunity for ASTM. The employment of fracture mechanics principles should help in achieving these objectives. The closer we come to this goal the more practical can be the policies that are issued by the HSB. This will facilitate compliance and also enhance durability and reliability. A shortening of testing time will also quicken the qualifying of new formulations and it will simplify policies on formulation changes that can be made to a listed formulation.

In regards to ASTM Committee F17's involvement in the pursuit of the above objectives this observer wishes to express a concern. When the first standards for thermoplastic pressure pipe were being written it was the policy of ASTM to write standards that only defined pipes based on requirements for material, dimensions and test results. This was in line with what ASTM then signified, the American Society for Testing and Materials. It was other organizations which were dedicated to issues of engineering design—most often a code body—that determined the value of an engineering parameter such as design stress and pipe pressure rating. This is why the earliest ASTM standards for thermoplastic pressure pipe list in an Appendix (a non-mandatory section) pipe design stresses and pressure ratings that are recommended by the PPI. Currently, Committee F17 is free to specify a design stress requirement and the method(s) based on which this stress is determined. The only obligation is that the proposal must receive the approval of the F-17 membership. But, expertise in engineering matters is not an F-17 membership requirement. This can result in less than a critical scrutiny of a proposal. Among its membership Committee F17 includes "experts" that may be promoting a standardization concept which is more advantageous to their client than to the advancement of optimum engineering. For the insuring that Committee F17 continues to issue the most technologically appropriate standards for thermoplastics pipe it is hoped that special attention is given to this matter.

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Long-Term Hydrostatic Strength and Design of Thermoplastic Piping Compounds

ABSTRACT: There has been tremendous growth in the use of thermoplastic piping systems since their introduction more than 50 years ago. They bring a host of benefits in the form of long-term performance and reliability, ease of installation, and not being prone to corrosion and tuberculation. It was clear early on that thermoplastics could not be evaluated in the same way metallic components would be in similar applications. However, over time the understanding of these materials has matured, and as this understanding continues to develop we must not lose sight of the evaluation methodologies used for establishing the long-term hydrostatic strength of these compounds, and how that strength has been successfully used in designing these systems. This paper will give an overview of the basic methodology used to establish the long-term hydrostatic strength of thermoplastic compounds, and how that strength is used for engineering design in a safe and reliable manner.

KEYWORDS: plastic pipe, strength, design, stress, thermoplastic

Introduction

The long-term strength of a thermoplastic compound cannot be determined from a short-term tensile strength test, as with most metals. As such, testing and evaluation methodologies have been developed which take into account not only the stress-rupture response of thermoplastics when subjected to only hydrostatic pressure, but that also take into account the potential changes in failure mode when subjected to stresses induced by other loadings than just hydrostatic pressure. This more comprehensive evaluation allows for the making of a more engineering appropriate forecast of the long-term strength of these materials so they can be safely used in a pressure pipe application [1].

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The common method for the forecasting of long-term strength relies on putting specimens under multiple continuous stress levels until failure. These data points are then used in a log-log linear regression evaluation. This regression equation is then extrapolated to a point sufficiently further out in time to where a long-term strength can be forecast.

It has been clearly established for many thermoplastics, including PE, that a failure mechanism which occurs at ambient temperature can be maintained and greatly accelerated by elevating the testing temperature. This acceleration has been shown to follow an Arrhenius, or rate process, behavior that is common to many chemical and mechanical processes. However, some thermoplastics, such as PVC, do not lend themselves to these types of accelerated testing methodologies, since they change phase at these elevated test temperatures and such testing would no longer be evaluating the same material properties. By testing at elevated temperatures it can be “validated” that the extrapolation remains linear and ductile beyond the actual test data. This and other criteria established by ASTM D2837 and the Plastics Pipe Institute’s Hydrostatic Stress Board policies in Technical Report-3 (TR-3) allow for establishing an appropriate maximum working stress that will assure a very long design life—well in excess of the stress regression extrapolation time [2,3].

Establishing the Long-Term Hydrostatic Strength

There are two primary methodologies established for determining the long-term hydrostatic strength of a thermoplastic material for a piping application—ASTM D2837, “Standard Test Method for Obtaining Hydrostatic Design Basis for Thermoplastic Pipe Materials or Pressure Design Basis for Thermoplastic Pipe Products,” and ISO 9080, “Plastics piping and ducting systems — Determination of the long-term hydrostatic strength of thermoplastics materials in pipe form by extrapolation” [4]. In addition, there is also a similar methodology used for thermoset compounds with fiber reinforcement—ASTM D2992 [5]. These methodologies are based on similar principles of evaluating the compound under constant stress—below the short-term tensile yield stress—in the form of a pipe and allowing the stress rupture response to be determined under these various stress loadings and durations. Using a single or multiple linear regression model, a forecast of the long-term hydrostatic strength of the material can be made. This is a similar approach to that used with metals in high temperature applications, such as boiler tubes. However, these boiler tubes are only exposed to hydrostatic pressure during service rather than the other potential stresses experienced in a buried piping application. Additionally, these regression curves for metals are a result of a different mechanism than for plastics and cannot be directly compared or applied in the same manner. The results from both the ASTM and ISO methods are valid, but it is ultimately the application of the appropriate design factor to establish a safe maximum working stress that is the key. In North America, ASTM standards are the dominant methodologies utilized, so this method will be further examined. A more detailed analysis of the differences in the ISO and ASTM methodologies are provided in a reference [6].

In the 1950s designing with thermoplastics was relatively new and there were no standardized methods to do so in a consistent and reliable manner. In 1958, the Thermoplastic Pipe Division of the Society of the Plastics Industry (subsequently named the Plastics Pipe Institute) established the Working Stress Subcommittee, the predecessor of the Hydrostatic Stress Board. This board consisted of various technical persons well-versed in the evaluation and forecast of the long-term strength of plastics. After studying the application for several years the first tentative method was developed, and in 1963 the group issued its first hydrostatic design stress recommendations for thermoplastic compounds. After evolving through fifteen iterations, this method was published in 1969 as ASTM D2837. The methodology has proven pertinent to all thermoplastic materials, and even thermoplastic based composite pipes, that exhibit a response of decreasing rupture strength when subjected to a continuous load.

There are several premises associated with ASTM D2837. It is important to understand that although the material is being evaluated in the form of a pipe, the result is not a “pressure” rating on the pipe, but rather a long-term hydrostatic strength of the compound. It is not a trivial idea that a circumferential stress is being induced in the material after it has been formed into a cylindrical shape using traditional extrusion techniques. Due to the effects of molecular orientation in the extrusion direction, the application of the major stress in the circumferential direction is considered to be the “worst case” condition for evaluation of the material.

The average value of the stress that is generated in the thermoplastic pipe is calculated using a thin-wall pressure vessel equation which takes the stress at the mid-wall of the pipe. Since most thermoplastics have a relatively low elastic modulus (compared to metals) it is considered that this method is appropriate for even heavy wall pipe

$$S = \frac{P(D - t)}{2t} \tag{1}$$

where: S = Stress, psi

P = internal pressure, psig

D = average o.d., inches

t = minimum wall thickness, inches

The D2837 test method requires that a minimum of 18 pipe specimens are placed on hydrostatic test at various stress levels so as to produce failures over at least 3 log decades in time (Table 1).

The data are analyzed by linear regression using a linear least squares approach to yield a best fit log-stress versus log-time straight line equation

$$h = a + bf \tag{2}$$

where: h = logarithm of failure time, hours

f = logarithm of failure stress, psi

Even though it is traditional to plot log stress (σ) on the y-axis and log time (t) on the x-axis, the D2837 regression calculation is performed in the

TABLE 1—Minimum data point distribution for ASTM D2837.

Hours on Test	Number of Failure Points
<1000	At least 6
10–1000	At least 3
1000–6000	At least 3
After 6000	At least 3
After 10,000	At least 1

traditional manner where stress (σ) is the independent variable for the least squares calculation. Conversely, for the evaluation of thermoset compounds, ASTM D2992 performs the linear regression calculation using time as the independent variable. This D2837 methodology yields a somewhat lower forecast of the long-term hydrostatic strength. While different from D2837, this forecast can still be used for design purposes as long as the chosen design factor adequately takes this into account.

The straight line equation, on a log-log scale, is used to extrapolate the expected strength to the 100 000 h intercept. This stress value is called the long-term hydrostatic strength (LTHS). While it is convenient to analyze this stress-rupture data in a log-log scale, it is apparent that the creep response slope is not linear, as shown in Fig. 1, with Cartesian coordinates. It can be seen that very early failures that are more indicative of a quick burst type test result in stresses that approach the tensile yield stress obtained by ASTM D638 using tensile bars

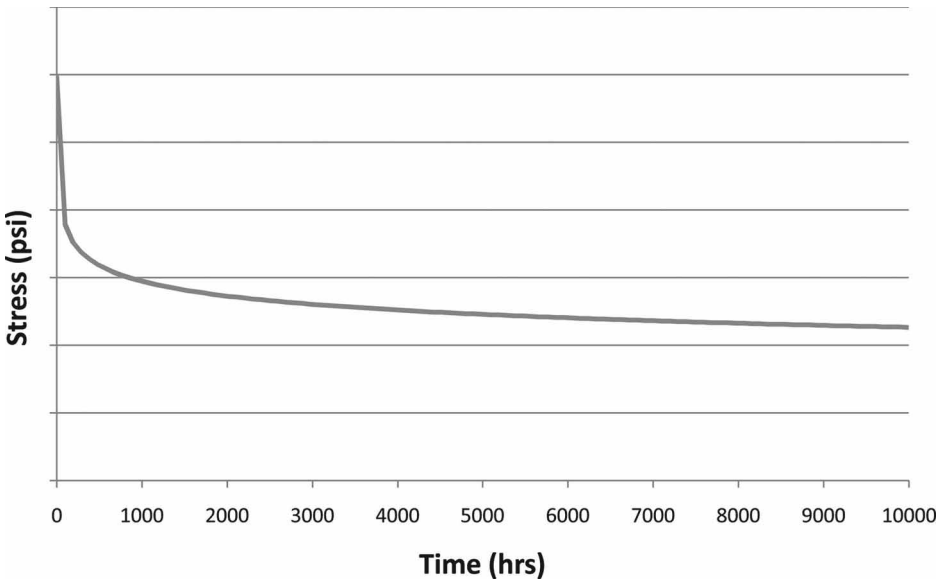


FIG. 1—Stress rupture data plotted on Cartesian coordinates.

with a strain rate around 2 in./es/minute. The curve starts to flatten out rather quickly; normally around one to ten hours depending on the type of material, and beyond 100 000 h the curve has become rather flat and approaches an almost steady-state.

Because of this near asymptotic response, and the imposed limitation of extrapolating to only one log decade in time beyond the actual test data, it was determined that extrapolation of the stress-regression data from 10 000 h of actual test data to 100 000 h is an appropriate time for establishing the long-term hydrostatic strength of the material. It should not be mistaken that the extrapolation time is in any way indicative of the design life or expected service life. With the application of the design factor, the design life will be well in excess of this intercept. The extrapolation time could have been extended beyond the 100 000 intercept (as is done in ISO 9080), but there were several important “safeguards” put into place to account for the potential of a lower long-term strength in other ways.

Additional Criteria

As stated in the scope of D2837, this method is applicable to materials for which the log-stress versus log-time-to-fail relationship is expected to continue through at least the 100 000 h intercept. This is an important requirement because if a change in slope, or downturn of the slope, were to occur prior to 100 000 h, this can result in two adverse consequences: (1) a forecast of a falsely greater long-term hydrostatic strength, and (2) a decrease of the strain at failure, which is particularly important because a lower strain capacity has been linked to a larger tendency to fail by the development and slow growth of cracks. For this reason, many metal material specifications require that the material exhibit a greater than 10 % ultimate strain during tensile testing.

There are several important caveats within the D2837 standard methodology to ensure that using the 100 000 h stress intercept value is appropriate even though the expected design and service life are significantly beyond this timeframe.

1. LCL/LTHS ratio >85 %: First, there is a statistical test to make sure the data is suitable for analysis by this method. The 95 % two-sided lower confidence limit (LCL) is calculated. If this LCL value differs from the LTHS value (i.e., mean value) by more than 15 %, the data is considered unsuitable for analysis by this method. This assures that there is no excessive scatter in the data and that using the mean strength value at 100 000 h is a sufficient estimation of the long-term strength of the material. It was also initially thought when the methodology was first developed that if a material did exhibit a change in the regression slope, or “knee,” prior to 10 000 h it would not pass this test. For thermoplastic materials that are known to exhibit a potential change in slope, other criteria have since been instituted in the form of elevated temperature hydrostatic testing to assure this is not the case, which is discussed later in more detail.
2. One log decade extrapolation: It is considered that extrapolating to one log decade beyond the actual test data was as far as should be done for statistically significant results.

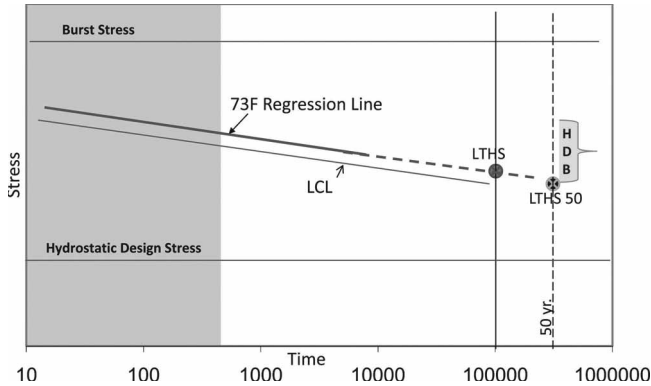


FIG. 2—Typical 73°F (23°C) regression line and extrapolation. Generalised ASTM D2837 73°F (23°C) regression line and extrapolation.

3. $LTHS_{50}/LTHS > 80\%$: To account for materials that have a steeper regression slope, it is required that if the 50 year intercept value is less than 80% of the 100 000 h intercept value (LTHS) then the more conservative 50-year value must be used as the long-term hydrostatic strength for design purposes. Again, this assures that the 100 000 h intercept value is an appropriate forecast and fairly represents the long-term strength of the material.
4. Limited circumferential expansion: The stress that results in 5% circumferential expansion at 100 000 h is to be considered the LTHS value. This limitation is not typically used for modern thermoplastic compounds used in piping applications.

Figure 2 is a graphic representation of a typical stress regression curve at 73°F.

Summary Steps

1. Linear least squares evaluation—2 coefficient model (Eq. (2))
2. Calculate 100 000 h strength—LTHS
3. Calculate 95% LCL— $LCL/LTHS > 0.85$
4. Calculate 50 year strength— $LTHS_{50} > 80\% LTHS$

Categorization of the LTHS

Once the long-term hydrostatic strength is established, this value is then categorized into ranges that are based on an R10 preferred number series of steps (also called a Renard Series after Charles Renard who developed the system in the early 1900s)—each base value for that category range is increased approximately 25% for the next window, or category. This is a common method for standardizing values so that each material does not have its own unique LTHS, but is placed in categories with other similar materials with at least minimum performance characteristics (Table 2).

TABLE 2—Hydrostatic design basis categories from ASTM D2837.

Range of Calculated LTHS Values		Hydrostatic Design Basis	
psi	(MPa)	psi	(MPa)
190 to <240	(1.31 to <1.65)	200	(1.38)
240 to <300	(1.65 to <2.07)	250	(1.72)
300 to <380	(2.07 to <2.62)	315	(2.17)
380 to <480	(2.62 to <3.31)	400	(2.76)
480 to <600	(3.31 to <4.14)	500	(3.45)
600 to <760	(4.14 to <5.24)	630	(4.34)
760 to <960	(5.24 to <6.62)	800	(5.52)
960 to <1200	(6.62 to 8.274)	1000	(6.89)
1200 to <1530	(8.27 to <10.55)	1250	(8.62)
1530 to <1920	(10.55 to <13.24)	1600	(11.03)
1920 to <2400	(13.24 to <16.55)	2000	(13.79)
2400 to <3020	(16.55 to <20.82)	2500	(17.24)
3020 to <3830	(20.82 to <26.41)	3150	(21.72)
3830 to <4800	(26.41 to <33.09)	4000	(27.58)

Validation and Substantiation

For some thermoplastic materials, elevated temperature accelerated testing has shown there may be a potential for a downturn in the slope of the stress regression response prior to 100 000 h. If this is a known potential, then it must be further proven that the extrapolation of the regression line beyond the actual test time is “valid.” This process is called “validation.” After establishing the 2-coefficient log-log equation and the appropriate LTHS, it now must be validated that the extrapolation of this line beyond the actual test time frame of 10 000 h is appropriate. While not seen in standard 73°F testing, it has been determined that PE materials have two potential failure mechanisms—ductile and slow crack growth, sometimes referred to as a “brittle” failure, which is not a failure mechanism unique only to PE. The point on the stress regression curve where it transitions from ductile type failures to brittle type has been termed the “knee” due to the change in slope of the regression curve. For the regression model calculations, ASTM D2837 requires that only failures that lie on the same line, or slope, can be utilized in the calculations to establish a LTHS at 73°F. For some other materials, such as PVC and CPVC, there does not appear to be a change of slope, even if the failures are deemed either “ductile” or “shatter” type failures. These two different failure mechanisms appear to fall on the same slope, so they are not differentiated by this evaluation methodology, even though a “shatter” type failure is usually the end result of crack growth. The development and growth of a crack occurs not in response to the average value of the imposed stress (such as from internal pressure), but due to the magnitude of a very localized stress that is greater than the average stress. Because of this behavioral difference, the design of ductile behaving materials can be based on

average stress. However, the design of brittle behaving materials must give consideration to the effect of localized stress intensifications. This consideration is the reason why the strength reduction factor for brittle materials is larger than that for ductile behaving materials.

Ductile failures consist of typical mass bulk deformation and high levels of plastic straining before failing in a burst fashion. Slow crack growth failures are small slitlike failures and are not accompanied by any apparent bulk deformation or strain in the pipe wall. These failures are typically very small and run axially along the pipe and not circumferentially. This is a very long-term failure mechanism that is initiated by a highly localized stress concentration in the pipe wall. Some causes may be an inhomogeneity in the pipe wall, or a contact loading on the outer surface of the pipe, such as a rock impingement. As the understanding of thermoplastic materials has progressed it is known that some, such as PE, PEX, PP, and PA, have failure mechanisms which behave in an Arrhenius response with temperature with essentially constant activation energy over the relatively small temperature range for testing. This has led to the development of very valuable testing methodologies that use elevated temperatures to accelerate both ductile and brittle failure modes so that the actual performance at lower temperatures can be forecast. Figure 3 shows multi-temperature regression curves at 73, 140, and 176°F with forecasts for both ductile and brittle failure slopes of early generation PE materials.

Validation of the Extrapolation

The rate process method is one such protocol frequently used for validating the extrapolation of the stress rupture curve [7]. The Plastics Pipe Institute Technical Note 16 (TN-16) details the application of the rate process method to PE materials [8]. With these accelerated “validation” methodologies, it can be forecast with even greater confidence that the transition from ductile to brittle, or “knee” is sufficiently far out in time in order to not be a factor in determining the HDB or HDS for the PE material. In recognition of this potential, there are specific tests that must be performed to assure that this ductile/brittle transition does not occur before 100 000 h. This is “validating” that the extrapolation of the linear regression model does in fact remain linear to 100 000 h. In the case of higher performing PE materials it is required that the ductile regression line remains linear through more than 50 years—which is called “substantiation” of the regression linearity. This becomes important for the application of an appropriate design factor used to reduce the HDB to an appropriate working stress.

Because of polyethylene’s Arrhenius response the accelerating effects due to temperature, elevated temperature testing protocols have been developed in ASTM D2837 and PPI’s TR-3. For instance, ASTM D2837 requires that the HDB at 73°F must be “validated” by testing specimens at 176°F (80°C) for 200 h without any failures. Furthermore, PPI’s TR-3, Part F.5 provides that the HDB forecast at 73°F can be substantiated to not have a “knee” prior to 50 years by testing pipe specimens at 176°F (80°C) to at least 6000 h without brittle failures.

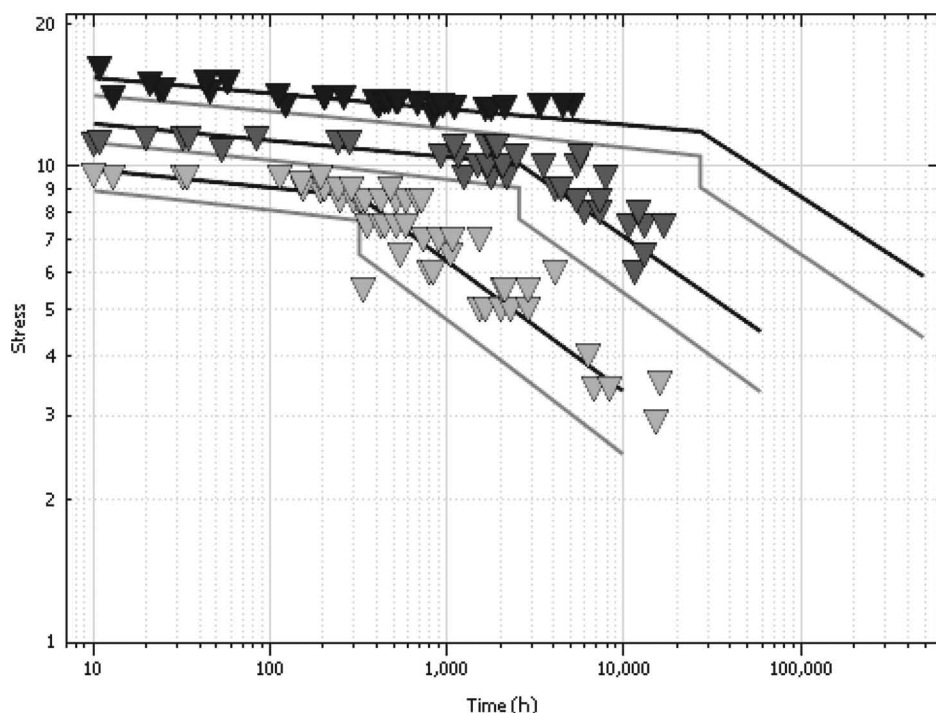


FIG. 3—Early generation (circa 1970s) PE materials showing forecast of ductile/brittle transition (lines depict the mean and the 95 % LCL curves).

PPI TR-3 Policies to Obtain a HDB

The PPI Hydrostatic Stress Board (HSB) uses ASTM D2837 to establish a recommended HDB. The policies for doing so are detailed in Technical Report-3, TR-3. The HSB requirements go beyond just a single D2837 evaluation by requiring that at least three individual lots of the subject material be tested. One of these lots must undergo the full 10 000 h testing according to D2837, while the additional two lots undergo “truncated” testing to less than 10 000 h (typically 2000 to 6000 h) in order to show consistency in material performance between production batches.

Once the evaluation is verified to meet the full requirements of TR-3, the recommended HDB at each temperature is published in Technical Report-4, TR-4. These material listings are recognized by various product standards and third party organizations, as well as the Federal DOT, as “proof” of the materials recommended hydrostatic design basis and hydrostatic design stress.

Design Factor and Hydrostatic Design Stress

Once the HDB has been determined for a thermoplastic compound, it is necessary to then reduce this strength into an allowable working stress (i.e., the stress

induced only by the internal pressure) for a long-term design in a way that will assure an indefinite design life with a satisfactory margin of safety, even when stresses other than those induced from internal pressure exist on the piping system (e.g., soil loads, bends, joints, rock impingement, scratches, gouges, etc...). This maximum allowable stress, or the hydrostatic design stress (HDS), is derived by multiplying the HDB by a strength reduction factor called the design factor (DF), which should not be confused with the traditional safety factor

$$\text{HDS} = \text{HDB} \times \text{DF} \quad (3)$$

where: HDS = hydrostatic design stress, psi

HDB = hydrostatic design basis, psi

DF = design factor, a number less than one

Understanding the basis and assumptions used for establishing the HDB, as along with the typical installation practices, operating conditions, and associated potential failure mechanisms of the material, is critical to establishing an appropriate reduction factor to determine the hydrostatic design stress. The definition of the HDS in ASTM D2837 is—"The estimated maximum tensile stress the material is capable of withstanding continuously with a high degree of certainty that failure of the pipe will not occur. This stress is circumferential when hydrostatic water pressure is applied." While the appropriate design factor is not given in D2837, notes 8 and 9 give some guidance for the determination of the appropriate design (service) factor.

Some aspects that must be considered when determining the appropriate design factor (DF):

1. Use of the mean strength forecast rather than the minimum, or LCL, value. It is required that the LCL be at least 85 % of the mean. This supports the idea that using the mean value is appropriate and not overly optimistic, but still must be taken into account with the design factor.
2. Taking the strength at 100 000 h rather than 50 years or longer. The design life of the system will normally be in excess of 50 years. If the 50 year value is less than 80 % of the 100 000 value, then use the 50 year value. This takes into account material with a more severe stress rupture slope, and is rated in a more conservative manner, if that is the case.
3. Potential variations in the material and pipe manufacturing process. This can be more critical for those materials that depend on the quality of extrusion to develop long-term strength. These variations will have some effect on the long-term performance of the piping system.
4. The failure mechanism—for some materials it is required that the HDB at 73°F is based solely on the ductile failure extrapolation curve. It must be validated through elevated temperature testing that the ductile/brittle transition, or "knee," is beyond 100 000 h. For high performance PE materials the HDB is substantiated to 50 years. For other materials that cannot be easily tested at elevated temperatures the linear continuation of the extrapolation must be assumed, which will also affect the appropriate DF.

5. Additional additive stresses the material is likely to encounter in a piping system from installation, soil loads, and typical operating conditions. Viscoelastic materials can undergo stress relaxation over time which can lessen the impact of these additive stresses.
6. How the material will respond to localized stress intensifications from imperfections in the pipe, rock impingement, notches, tapping, soil shifts, etc... A more ductile material will shed these intensified stress point loadings, while a more brittle material is likely to be damaged, resulting in crack growth and early failure.

When the Working Stress Committee first developed this methodology it was decided by consensus that a “satisfactory” design factor for thermoplastics in the piping application should be 0.5 (note: it was initially a divisor of 2.0, but in order to differentiate the design factor from a safety factor it was changed to a multiplier of 0.5). There were arguments for both a lower and higher value. This is considered an “overall” factor that can be applied to most operating conditions considered to be routine. The design engineer is always able to use additional reduction factors if deemed necessary for the application.

This reduction factor was deliberately termed a *design factor* rather than a *safety factor* because the way it is being applied is not a true safety factor. It is understood that the ratio of the material’s long-term strength (i.e., HDB) to the actual induced stresses from internal pressure was not 2:1, nor did it need to be. There is a large margin of safety (greater than 2:1) against failure from over pressurization or the slow growth of a crack over the service life of the piping system.

It must also be understood there is not a “strength reservoir” for most thermoplastic materials that is being drawn from when exposed to a continuous long-term stress condition. Stress rupture strength is not a measure of the strength of the material at a particular point in time, but rather the response to a constant load over a long period of time. When the load is reduced or removed its effect is altered. The rate of creep, a viscoelastic response, is diminished or reversed. If the material is operating at a condition where “brittle” type failures are likely to occur, this rate is also diminished or stopped. There is little, if any, residual effect. This will be slightly different for various types of thermoplastics depending on their viscoelastic response to stress and strain.

Research has shown that for PE pipes that have been under constant hydrostatic pressure for more than 100 000 h (eleven years) at a temperature of 140°F (60°C) there is no decrease in the burst strength of that pipe [13]. Table 3 summarizes this research. PE pipe compounds A through G were tested under constant hydrostatic stress according to ASTM D1598 at a temperature of 140°F (60°C) for up to 115 000 h [9]. After being removed from test without having failed, they were subjected to quick burst testing as per ASTM D1599 [10]. As can be seen from the resulting data there was no decrease in the burst pressure of the tested pipes as compared to the control pipes which were not tested under continuous hydrostatic stress. Applying the estimated Arrhenius response for temperature acceleration, testing at 140°F (60°C) for 100 000 h is equivalent to more than 300 yr at 73°F (23°C). This is a key reason why the design factor (DF), as applied to thermoplastic pipe design, is not simply the inverse of the

TABLE 3—Research depicting burst strength of PE pipes—control versus aged.

Formulation	Applied Hoop Stress During Long-Term Testing, psi	Time on Long-Term Test at 140°F (hours) by ASTM D1598	Average Burst Pressure, psig, by ASTM D1599	Burst Quality
A (R398)	Control	0	945	Ductile
A (R398)	700	115751	935	Ductile
B (R443)	Control	0	1144	Ductile
B (R443)	719	112840	1112	Ductile
C (R834)	Control	0	1327	Ductile
C (R834)	826	52896	1202	Ductile
D (R761)	Control	0	1292	Ductile
D (R761)	997	61488	NA ^a	Ductile
E (R853)	Control	0	1287	Ductile
E (R853)	853	46153	1205	Ductile
F (R833)	Control	0	1397	Ductile
F (R833)	829	52940	1267	Ductile
G (R881)	Control	0	1299	Ductile
G (R881)	798	42196	1249	Ductile

NA: indicates data not available.

safety factor. During the entire service life of a thermoplastic piping system the margin of safety against failure from short-term over pressurization will always be greater than two to one (2:1)—actually approaching four to one (4:1) in many cases. For instance, a 24 in. SDR 11 PE 4710 pipe may have a maximum operating pressure of 200 psig at 73°F. The short-term burst pressure of this pipe would be around 1000 psig, even after many, many years in service.

Effect on Design Life

Historical field performance over the decades has shown that it is not the hoop stress from internal pressure that will determine the service life of a plastic pipe, but rather how the material responds to other induced stresses during service and the corresponding stress intensification (point loading, fatigue, surge, etc...) [11]. Ductile materials will shed these stress concentrations by distributing the stress into the surrounding matrix, while more brittle materials will tend to be further affected and result in a reduced service life. This is why ductile materials are typically assigned a higher strength reduction factor than more brittle behaving materials.

Since the application of the design factor is typically under the purview of the design engineer and was not part the ASTM standards process, the Plastics Pipes Institute Hydrostatic Stress Board has been the source of a recommended hydrostatic design stress (HDS) and thus the corresponding DF for thermoplastic materials intended for piping applications for more than 40 years. The application of the strength reduction factor (i.e., design factor) results in a maximum

hydrostatic design stress (HDS) which takes into account that there will be additional stresses on the system, as well as how the piping material will respond to these conditions as potential failure mechanisms. This has been a very successful design practice. These HDS recommendations for thermoplastic compounds are published in the Plastics Pipe Institute Technical Report-4 (TR-4) [12].

Moving Past the 0.50 Design Factor with Enhanced Performance Criteria

After several years of studying the subject of potentially increasing the recommended maximum working stress for some PE materials, the Hydrostatic Stress Board (HSB) determined that, based on the historically proven field failure modes, if the material demonstrated sufficient resistance to failure induced by localized stress intensifications (i.e., slow crack growth resistance), which is the main failure mechanism for plastic piping systems, that it could be operated at a somewhat higher bulk stress induced from internal pressure without sacrificing the overall operating margin of safety of the piping system. This concept was further supported by the successful 20 yr operating history of these types of PE piping systems designed and operated under the ISO series of standards which allow for a design stress as high as 1160 psi. Historical recommendations from the HSB limited these materials to a recommended maximum operating stress of 800 psi.

At the end of its investigation the HSB established three critical performance criteria for a PE material to qualify for the higher 0.63 design factor; in addition to those requirements already imposed by ASTM D2837 and PPI TR-3. PE materials not meeting these requirements continue to utilize the 0.50 DF for determining the maximum recommended design stress at 73°F.

Additional required criteria for PE materials in order to be utilized with a 0.63 design factor:

1. A demonstration, in accordance with Plastics Pipe Institutes policies in TR-3, that in its stress-rupture evaluation by D2837 a PE material shall continue to fail by the ductile mode through at least the 50-year intercept at 73°F. This requirement ensures that the PE material continues to operate in the ductile state even after very prolonged periods of sustained stress.
2. The 95 % lower confidence limit (LCL) of the projected average value of the material's long term hydrostatic strength cannot be less than 90 % of the average value that is forecast by means of method ASTM D2837 (i.e., the LCL/LTHS ratio must be great than 0.90). A high LCL/LTHS ratio enhances the statistical reliability of the forecast of the long-term strength. It is also another indication of high resistance to failure by a brittle-like mechanism, a mechanism that leads to greater scatter in stress-rupture data.
3. The minimum failure time under ASTM test method F1473, a fracture mechanics based test method that uses a combination of a significantly elevated test temperature and a very sharp notch that induces a high

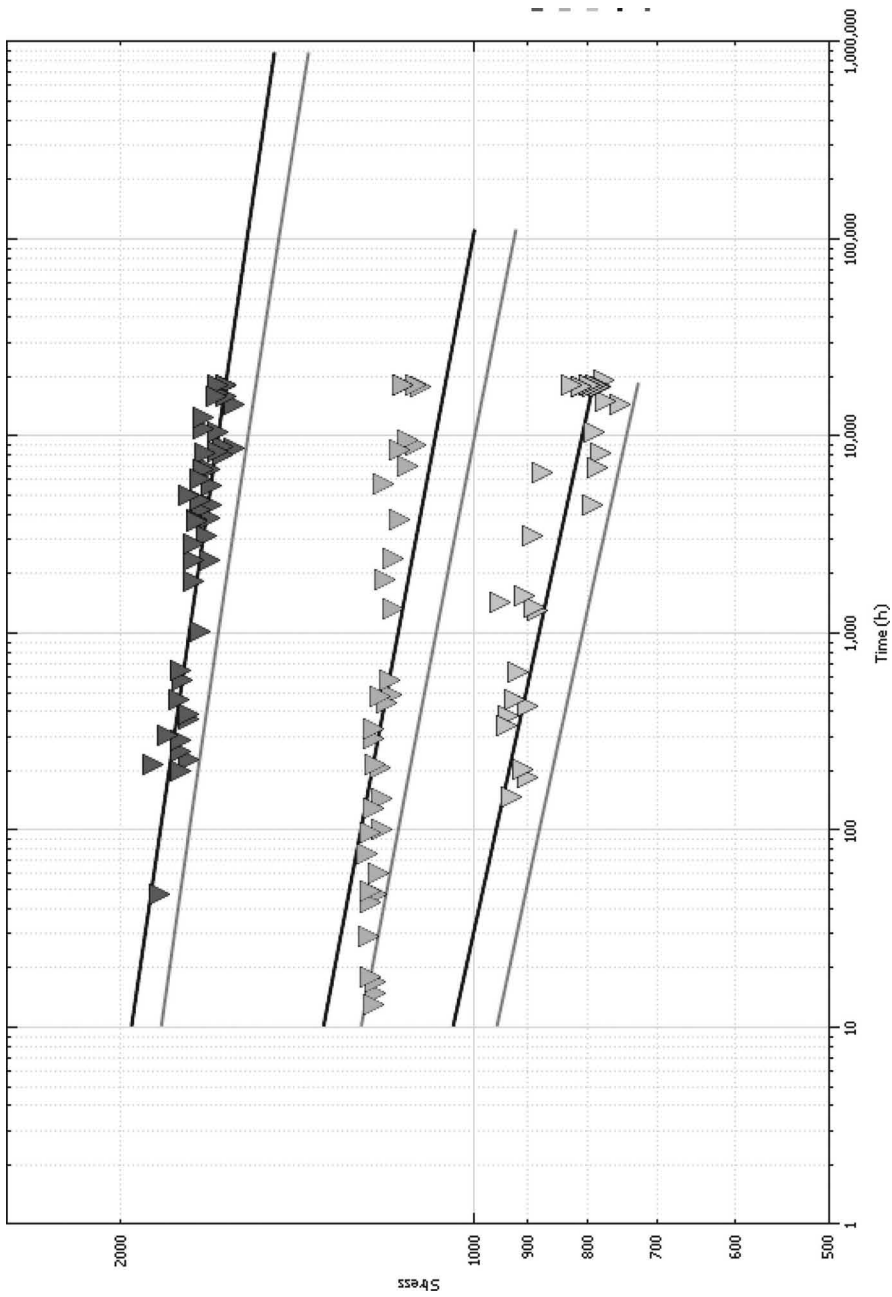


FIG. 4—Multi-temperature (73, 140, and 176°F) stress rupture curves of high performance PE material where all failure points are ductile in nature.

localized stress intensification in the test specimen, cannot be less than 500 h. This time is about five times greater than the test time which, based on a calibration of test results versus actual field experience, has been found to result in essential immunity to the effect of localized stress intensifications that occur under actual field conditions.

If a PE material meets these enhanced performance criteria, it is appropriate to be able to operate at a higher maximum design stress induced by internal pressure because the additive stresses during buried piping service will be much less likely to result in localized stress intensifications that could cause a brittle failure [13]. As compared to early generation PE materials for piping applications depicted in Fig. 3, stress regression curves for high performance PE materials meeting the additional criteria are shown in Fig. 4. These curves are the result of entirely ductile failure points. It can be seen that the transition to brittle failure, indicated by a downturn in the slopes of Fig. 3, has been essentially eliminated as a potential failure mechanism at service temperatures and conditions, so this material qualifies for the higher HDS and corresponding working pressure rating. Even at this higher operating pressure, there remains a substantial "safety factor" against ductile rupture due to short-term over pressurization. Of course the question could be asked, "Why 0.63 rather than 0.67 or some other value?" The answer comes from ASTM D2837. It is recommended that when applying a service design factor the number is selected from ANSI Z17.1-1973 or ISO 3 for Preferred Numbers in the R10 series. In the R10 series 0.63 is the next step up from 0.5. The HSB studied what the higher allowable operating stress would be and determined that a PE material could perform safely at this level compared to the burst stress and expected service life.

Conclusion

Unlike metals, thermoplastic materials cannot have their long-term strength determined from a short-term tensile test. However, like metals, the quality and length of structural performance is greatly determined by the material's continuing capacity to operate in the ductile state. Due to viscoelastic behavior the response under constant steady-state loading must be determined by applying a constant load at various stress levels in order to produce failures ranging over several log-decades in time as per ASTM D2837, or other similar stress regression methods. Once the LTHS of the material has been established it is categorized into a hydrostatic design basis for standardization purposes. This HDB must then be reduced to a maximum working stress induced by internal pressure (HDS) with the application of a design factor. In determining the appropriate design factor, many considerations must be taken into account—the methodology used to determine the long-term strength and associated assumptions, along with the potential failure mechanisms of the material and the environment in which it will be used. Results of field experience has shown that the nature of the mechanism by which a thermoplastic pipe material fails under actual service conditions is a most important factor in the establishment of that material's design stress for pressure pipe applications. In North America, for thermoplastic materials analyzed by D2837 the PPI Hydrostatic Stress Board

has recommended a design factor of 0.5 as the historical reduction factor. Other parts of world have long used a less conservative factor, as high as an equivalent of 0.71, and have had a very successful operating history associated with their particular installation and operating practices.

It is also important to note that evaluation methodologies used to determine the long-term hydrostatic strength do not adequately replicate the conditions and stresses a thermoplastic pipe is likely to experience during service. In the early 2000s the Hydrostatic Stress Board determined that high performance PE materials that meet stringent additional criteria can be operated at a higher hydrostatic design stress determined utilizing a 0.63 design factor, while still maintaining a large margin of safety against failure—both short-term and long-term. These materials remain ductile for a very long time and exhibit high resistance to slow crack growth, and thus are much less prone to failure from localized stress intensifications which can occur during normal operation of a plastic piping system. Systems designed under this methodology will continue to have a design life well in excess of 50 years.

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NSF 14: Shaping the Future of the Plastic Piping Industry

ABSTRACT: In 1951, NSF, then the National Sanitation Foundation, was approached by the Thermoplastics division of the Society of the Plastics Industry to initiate a research study related to the suitability of plastic pipe for potable water supplies. The results were reported in 1955 under the title “A Study of Plastic Pipe for Potable Water Supplies.” Following the publication of the study, NSF was requested by the plastics industry to establish a continuing testing and certification program on plastics materials, pipe, and fittings for potable water applications not only for toxicological considerations but also for performance evaluation. The Public Health and Industry Advisory Committee recommended the use of ASTM test procedures. The certification program was first started in 1959 and led to the creation of National Sanitation Foundation Standard Number 14 for Thermoplastic Materials in 1965. In 1991, NSF 14 became ANSI/NSF 14, the national standard for Plastic Piping Systems Components and Related Materials.

KEYWORDS: NSF 14, standard, certification, plastic, pipe

Introduction

The National Sanitation Foundation, known today as NSF International, was created in 1944 in the School of Public Health of the University of Michigan by three professors, Walter Snyder, Henry Vaughn, and Nathan Sinai. Their goal was to create “an independent organization which would act as a liaison between the industry and health authorities in order to find solutions to modern sanitation problems affecting both the industry and the public health. The Foundation was a non-profit, non-commercial organization dedicated to the prevention of illness, the promotion of health and the enrichment of the quality of living.”

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In 1952, the council of Public Health Consultants was created together with the Joint Committee on Food Equipment Standards, leading to the publication of the first NSF standard, NSF 1, Soda Fountain—Luncheonette Equipment and Appurtenances [1]. In 1960, NSF expanded its scope to include swimming pool equipment by publishing NSF 9, Standard No. 9 Relating to Diatomite Type Filters for Swimming Pool Equipment [2]. In 1963, the Joint Committee on Thermoplastics Standards was created following the publication of the study related to the suitability of plastics for potable water distribution. It was composed of public health officials and industry representatives. The goal of this committee was to develop NSF 14, National Sanitation Foundation Standard for Thermoplastic Materials, Pipe, Fittings, Valves, Traps and Joining Materials [3].

This paper will present the findings of the “Study of Plastic Pipe for Potable Water Supplies,” [4] the document that led to the creation of Standard Number 14. It will then go over some of the main requirements included in the first issue of NSF 14 published in 1965. The evolution of NSF 14 through the main additions will also be examined. Finally, it will give an overview of the current ANSI (American National Standards Institute)/NSF 14 standard and discuss its contribution to the plastics industry.

A Study of Plastic Pipe for Potable Water Supplies

In 1951, NSF received a research grant from the plastics industry through its Thermoplastic Pipe Division to evaluate the suitability of plastics pipe for potable water supplies from the public health point of view. This study resulted from the growing interest in using plastic pipe for underground potable water distribution because of its many advantages, including ease of installation, resistance to chemical attack, light weight, and lower costs.

A synopsis of the study, including sample selection, test methodology, and results is presented in the following.

1. *Selection of test samples:* Twenty-two samples were selected, including commonly used polymers such as polyvinyl chloride (PVC), polyethylene (PE), and two polymers that are no longer used by industry, rubber-modified polystyrene (PS) and cellulose acetate butyrate (CAB). Two formulations unsuitable for potable water application were also included—sample 130, a PE compound used in electrical insulation and samples 280 and 290, PVC formulation containing small quantities of lead and cadmium.
2. *Test methodology:* The following tests were selected to determine the possible effect of plastic pipe on potable water and also the possible effect of water on the pipe itself. The test included air, water, and soil contact effects, toxicological aspects, practicability of disinfection procedures, extraction studies, taste and odor tests, effect of plastic pipe on chlorine residuals in tap water, effect of high concentration of chlorine on plastic pipe, and susceptibility of plastic pipe to attack by rodents.
 - *Exposure to water, air and soil:* Test specimens to evaluate the effect of water consisted of longitudinally cut pieces of pipe placed in air

sealed jars filled with Ann Arbor tap water. Test specimens to evaluate the effects of soil also consisted of longitudinally cut pieces of pipe placed in air sealed jars of soil of pH 2. The jars were kept in a 35°C incubator for 1 year. Specimens to evaluate the effect of natural weathering were suspended on a wire and hung on the roof of the School of public Health also for 1 year. After exposure, weight change was measured and visual inspection was performed.

- *Toxicological tests:* Test water was made by exposing Ann Arbor tap water adjusted to pH 5 for 3 days at 35°C to various types of plastics. The water was then fed to Wistar strain white rats for a period ranging from 6 to 19 months. During the test period, water consumption, food consumption, and growth of the rats were monitored. In addition, autopsies were performed at 6 month intervals on each rat group.
- *Experimental disinfection of plastic pipe systems:* Test systems consisting of each of the four groups of materials (PVC, PE, PS and CAB) were subjected to a specific disinfection procedure following contamination with *Escherichia Coli* (E. Coli). Samples were collected after disinfection with a hypochlorite solution and tested for presence of bacteria.
- *Extraction studies:* The first step in the extraction study was the selection of the different type of test waters. The waters had to be as aggressive as natural water. Due to their specific chemical composition, including high content of ingredients such as chlorides, sulfates, bicarbonates, iron, chlorine, fluoride and free carbon dioxide, five locations from Michigan—Ann Arbor, Clio, Dexter, East Lansing and Milan—and three from Illinois—Deland, Benton, and Enfield—were selected. Specimens of plastic pipe for the extraction test were prepared by first cutting 1/8 in. thick rings, which were then quartered. They were then placed in glass jars filled with 400 mL of each of the eight types of test water in order to give approximately 600 cm² of exposed surface. All jars were conditioned at 35°C for 72 h and then tested for color, turbidity, total dissolved solids, pH, lead, iron, nitrite, chlorides, and hardness. Extraction tests were also performed on PVC samples that contained higher level of lead stabilizer—5 % and 10 %.
- *Taste and odor:* Tastes and odors were determined on water exposed to the plastic pipe by using the extraction method, as well as on similar water recirculated through a system of plastic pipe; thus taking into account the effects of fittings and jointing compounds.
- *Effect of chlorine residuals:* In order to determine the effect of plastic pipe on chlorine residuals in tap water, a series of tests were performed. Five identical sets of each plastic were exposed to tap water containing 0.8 ppm total chlorine, 0.1 ppm active and 0.7 ppm combined chlorine while being kept in the dark at 10°C, for specific time periods—1, 3, 5, 24, and 48 h. After each time period, total residual chlorine content was measured by the orthotolidine-arsenite (OTA) test.

- *Effect of high concentrations of chlorine:* To determine the effect of high concentrations of chlorine on the various groups of plastic pipe, an experiment was setup by exposing whole short pieces of pipe to solutions containing 800–900 ppm of chlorine. Sample length, weight, and o.d. were measured before and after exposure.

3. Results.

- *Exposure to water, soil, and air:* Samples of plastic pipe exposed to natural weathering, soil burial, and water submersion for a period of a year showed only slight changes. The weathered and submersed samples showed some minimal change in color, while the buried samples showed some slight discoloration.
- *Toxicological test results:* Observations and tests performed on rats exposed to plastics showed no appreciable abnormal effects as compared to the control group. Additionally, the result of autopsies showed no evidence of any damaging action attributable to the test chemicals.
- *Extraction tests:* The result of the tests showed that it was impossible to extract any deleterious substances from any of the samples of pipe tested which were recommended by the manufacturers for potable water application. This was true even when the pH of the water was lowered to 5.0. However, the positive control samples—PE electrical insulation compound and PVC compound with high lead content—did show traces of colorant and lead.
- *Taste and odor:* The results showed no noticeable changes in odors and tastes in any of the samples exposed to plastic pipe than in the controls.
- *Disinfection of plastic pipe systems:* The results indicated that the plastic pipe systems can be satisfactorily disinfected following standard disinfection procedures. Experiment also showed that higher than usual concentrations of chlorine may be used in the disinfection of plastic pipe without damage.

Standard Number 14 on Thermoplastics Materials—1965

Following the publication of the research report in June 1955, NSF was requested by the plastics industry to establish a continuing testing and certification program on plastic materials, pipe, and fittings for potable water applications. At first, the certification program was only related to toxicological and organoleptic properties. However, it became evident that the program also needed to address the performance properties of the different products. Therefore, in 1959, the program for performance evaluation was created. American Society for Testing and Materials (ASTM) test procedures as specified in the Commercial Standards of the Commodity Standards Division, U.S. Department of Commerce, were used since there were no physical standards established by the American Water Work Association (AWWA). It also became necessary to create a document that would coordinate and contain the different requirements of the certification program. A joint committee on thermoplastics

standards, composed of public health officials and industry representatives, was created in 1963 to oversee the development of this new standard.

After two years of committee work, The National Sanitation Foundation Standard Number 14 on Thermoplastic Materials was published in 1965 [5]. Its purpose was to establish the necessary public health and safety requirements for thermoplastics materials, pipe, fittings, valves, traps, and joining materials based on specific end-use and application—potable water systems and drainage and vent systems. Some of the main requirements are listed in the following.

- Section 3. Materials.
 - 3.01 *Hydrostatic Design Stress*: It required materials for use in the manufacturing of plastic pipe for pressure application to submit evidence or data to permit the assignment of a design stress in conformance with Methods for Estimating Long-Term Strength and Working Stress of Thermoplastic Pipe. This method was developed by the Thermoplastic Pipe Division of the Society of the Plastics Industry based on a 4 year evaluation and testing program at Batelle Memorial Institute, Columbus, OH. One of the most important finding of the Batelle Program was that the short term burst test and 1000 h sustained pressure test could not provide the basic data necessary for the design of thermoplastics pipe. In 1966, this method was issued as ASTM D2837 [6] and dropped by SPI.
 - 3.02 *Potable water application*: This section required thermoplastic materials for potable water applications to meet the toxicological and organoleptic requirements of the Public Health Service Drinking Water Standards per the procedures of “A Study of Plastic Pipe for Potable Water Supplies.”
 - 3.04 *Materials produced by compounders*: Material produced by compounders had to meet additional quality control requirements:
 1. Biannual testing for toxicological and organoleptic properties.
 2. Reports on ingredient purchases and production numbers.
- Section 4 requirements for pipe, fittings, valves and traps.
 - 4.02 *Physical, performance, and dimension requirements*: Standard 14 required that all pipe, fittings, and valves comply with the physical, performance, and dimension requirements of the latest applicable Standards of the American Standard Association (ASA), American Society of Testing Materials (ASTM), or the Commodity Standards Division of the U.S. Department of Commerce (CS) such as
 1. CS 254-63, Acrylonitrile-Butadiene-Styrene (ABS) Plastic Pipe (SDR-PR and Class T) [7].
 2. CS 255-63, Polyethylene (PE) Plastic Pipe (SDR-PR) [8].
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 6. ASTM D2104-62, Polyethylene Plastic Pipe [12].
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 8. ASTM D2241-64, Polyvinyl Chloride (PVC) Plastic Pipe (SDR-PR and Class T) [14].
- 4.04 *Special engineered pipe, fittings, valves and traps*: Standard 14 allowed for new designs of pipe, fittings, and valves that did not meet the existing standards. It required special testing based on the end-use of the product. In addition, it mandated a special marking requirement “SE” to designate a specially engineered product.

The Evolution of Standard Number 14

Since its first publication in 1965, NSF 14 went through several important additions.

In 1971, section 6 “Quality control” and section 7 “Thread Compounds, Sealants and Lubricants” were added. Section 6 required manufacturers to provide and maintain quality control procedures at each production facility. Section 7 contained material and performance requirements for thread compounds, sealants, and lubricants. Section 7 required manufacturers to submit complete formulation of their product and notify the certifying agency of any changes in the amount or source of ingredient. Also for the first time, instead of just referencing an ASTM test method, NSF 14 now contained specific test methods such as short term rupture test, sustained pressure test, and breakout test.

In 1973, requirements for corrosive waste systems were added under section 8. Since there were no established consensus standards at that time for the performance requirements, NSF 14 viewed such systems as Special Engineered products falling under section 4.

In 1976, detailed quality control tables were added under Policy D. They contained critical dimension requirements and testing frequency based on the product and end-use type. It also contained frequency of performance testing requirements based on the material type.

In 1978, the title of NSF 14 was changed from Thermoplastics Materials to Plastics Piping System Components and Related Materials. It also saw the addition of section 9 and section 11 on marking and records, respectively. Section 9 required pipe to have marking addressing pressure rating, date code, and type of resin. Section 11 required manufacturers to keep records on all quality control tests and product code identification for a period of 2 years.

In 1984, section 12 on Plastics Ingredients was added. It contained specific toxicological requirements on ingredients used in PVC materials and compounds such as calcium carbonate, calcium stearate, hydrocarbon wax, and oxidized polyethylene wax. Ingredients also needed to comply with the Plastic Pipe Institute’s PPI TR-3, Policies and Procedures for Design Stresses for Thermoplastic Pipe Materials [15].

In June 1988, NSF Standard Number 61, Drinking Water System Components—Health Effects, was published [16]. This standard was developed at the request of the U.S. Environmental Protection Agency (EPA) in cooperation with the American Water Work Research Foundation (AWWARF), the Conference of State Health and Environment Managers (COSHEM), the AWWA, and the Association for Safe Drinking Water Administrators (ASDWA). Since it contained health effect requirements for any product in contact with potable water including plastic material, pipe, and fittings, all requirements pertaining to health effects were taken out of Standard 14 and replaced by referencing Standard 61.

In 1990, NSF Standard Number 14 becomes ANSI/NSF 14, the American National Standard for plastic pipe, fittings, and materials [17]. As a national consensus standard, it no longer contains the certification policies that were proprietary to the National Sanitation Foundation (NSF) Plastics Program such as the use of the NSF seals and logo or the inspection of manufacturer's facility.

In 2004, two new standards were added to section 2—ASTM D2513 [18] and Canadian Standards Association's CSA B137.4 [19], the American and Canadian standard for plastic pipe and fitting for natural gas application, respectively. This addition changed the perception that NSF 14 was only for water application. NSF 14 was now the national standard for all applications that involved plastic piping system.

ANSI/NSF 14 Plastics Piping System Components and Related Materials

Today's version of ANSI/NSF 14 is very similar to the first publication. Despite going through 20 revisions over its 45 years of existence, the same rigorous requirements as listed in the following remain:

- *Physical properties of compounds:* The material used in the fabrication of pipe and fittings needs to meet the applicable cell class requirements as referenced in the performance standards.
- *Performance testing:* Pipe, fittings, and solvent cement need to meet the applicable performance requirements standards as referenced in section 2. In addition to ASTM, NSF 14 also references CSA, International Standard Organization (ISO), AWWA, and Underwriter's Laboratory (UL) standards.
- *Long term hydrostatic requirements:* Materials for use in pressure applications need to show evidence of long term hydrostatic strength by either submitting data in accordance with PPI TR-3 or listing to PPI TR-4. NSF 14 allows both the ASTM D2837 and the ISO 9080 procedures as calculation method.
- *Potable water application:* Material, compounds, products, and formulations for potable water applications must comply with the applicable requirements of ANSI/NSF 61.
- *Quality assurance program:* NSF 14 requires the manufacturer of products listed to NSF 14 to have a quality control program in place in order

to show continuous compliance. Some of the requirements of this program are as follows:

1. Formulation verification.
2. Qualification of extruders and molds.
3. Calibration of testing equipment.
4. Quality assurance records.
5. Minimum number of test specimens for quality control testing.
6. Product specific quality control testing—type of test and frequency.
7. Ongoing monitoring program—annual testing.

These rigorous requirements were pivotal in initially establishing plastic as an acceptable material for use in both potable water and drain waste and vent application. They also contributed to the use of plastic piping system for corrosive waste and sewer application. Finally, they eased the acceptance of new plastic materials such as cross-linked polyethylene (PEX), polypropylene (PP), and polyethylene of raised temperature (PE-RT).

Conclusion

Since the publication of the results on the research study by the National Sanitation Foundation in 1951 that led to the creation of Standard 14 in 1965 to the current 2009 version, ANSI/NSF 14 has had a successful partnership with the plastics industry. For the last 45 years, it has helped shape the quality and safety of plastic pipe and fittings in the market place. Due to its long history and international recognition, NSF 14 has positioned itself as the premier standard for plastic piping system components and related materials.

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ASTM D 3350: A Historical and Current Perspective on a Standard Specification for Identification of Polyethylene Plastics Pipe and Fittings Materials*

ABSTRACT: ASTM D 3350, Standard Specification for Polyethylene Plastics Pipe and Fittings Materials, was originally approved and published in 1974 and has increasingly been referenced in numerous F17 Piping Application Standards. This paper/presentation will review the background of the original standard, provide how it has transformed as the industry has evolved to improved materials and “salt and pepper” blending for pipe production, and identify the challenges in modifying the standard to apply for all polyethylene piping applications. ASTM D 3350 is under the jurisdiction of ASTM Committee D20 on Plastics and is the direct responsibility of Subcommittee D20.15 on Thermoplastic Materials.

KEYWORDS: cell classification, D 3350, polyethylene, materials

Importance of ASTM D 3350 Standard Specification

In 1974, ASTM D 3350 standard specification was first published as a means for identification of polyethylene plastic pipe and fittings materials according to a cell classification system. Over the last 35 years, this standard specification has provided a foundation to identify materials for various polyethylene piping applications. This is evidenced by the fact that over 40 ASTM standard specifications in Vol. 8.04 reference this standard specification (see Table 1) along with various end-use industry specifications.

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TABLE 1—ASTM standards referencing ASTM D 3350 in Volume 8.04.

ASTM	Description
D 2104-03	Standard Specification for Polyethylene (PE) Plastic Pipe, Schedule 40
D 2239-03	Standard Specification for Polyethylene (PE) Plastic Pipe (SIDR-PR) Based on Controlled Inside Diameter
D 2447-03	Standard Specification for Polyethylene (PE) Plastic Pipe, Schedules 40 and 80, Based on Outside Diameter
D 2513-08	Standard Specification for Thermoplastic Gas Pressure Pipe, Tubing, and Fittings
D 2683-04	Standard Specification for Socket-Type Polyethylene Fittings for Outside Diameter- Controlled Polyethylene Pipe and Tubing
D 2737-03	Standard Specification for Polyethylene (PE) Plastic Tubing
D 3035-08	Standard Specification for Polyethylene (PE) Plastic Pipe (DR-PR) Based on Controlled Outside Diameter
D 3261-03	Standard Specification for Butt Heat Fusion Polyethylene (PE) Plastic Fittings for Polyethylene (PE) Plastic Pipe and Tubing
F 412-07	Standard Terminology Relating to Plastic Piping Systems
F 667-06	Standard Specification for Large Diameter Corrugated Polyethylene Pipe and Fittings
F 714-08	Standard Specification for Polyethylene (PE) Plastic Pipe (SDR-PR) Based on Outside Diameter
F 771-06	Standard Specification for Polyethylene (PE) Thermoplastic High-Pressure Irrigation Pipeline Systems
F 810-07	Standard Specification for Smoothwall Polyethylene (PE) Pipe for Use in Drainage and Waste Disposal Absorption Fields
F 894-07	Standard Specification for Polyethylene (PE) Large Diameter Profile Wall Sewer and Drain Pipe
F 1055-98	Standard Specification for Electrofusion Type Polyethylene Fittings for Outside Diameter Controlled Polyethylene Pipe and Tubing
F 1056-04	Standard Specification for Socket Fusion Tools for Use in Socket Fusion Joining Polyethylene Pipe or Tubing and Fittings
F 1281-07	Standard Specification for Crosslinked Polyethylene/Aluminum/Crosslinked Polyethylene (PEX-AL-PEX) Pressure Pipe
F 1282-06	Standard Specification for Polyethylene/Aluminum/Polyethylene (PE-AL-PE) Composite Pressure Pipe
F 1335-04	Standard Specification for Pressure-Rated Composite Pipe and Fittings for Elevated Temperature Service
F 1412-01	Standard Specification for Polyolefin Pipe and Fittings for Corrosive Waste Drainage Systems
F 1473-07	Standard Test Method for Notch Tensile Test to Measure the Resistance to Slow Crack Growth of Polyethylene Pipes and Resins
F 1533-01	Standard Specification for Deformed Polyethylene (PE) Liner
F 1606-05	Standard Practice for Rehabilitation of Existing Sewers and Conduits with Deformed Polyethylene (PE) Liner
F 1734-03	Standard Practice for Qualification of a Combination of Squeeze Tool, Pipe, and Squeeze-Off Procedures to Avoid Long-Term Damage in Polyethylene (PE) Gas Pipe

TABLE 1—*Continued*

ASTM	Description
F 1759-04	Standard Practice for Design of High-Density Polyethylene (HDPE) Manholes for Subsurface Applications
F 1901-04	Standard Specification for Polyethylene (PE) Pipe and Fittings for Roof Drain Systems
F 1970-05	Standard Specification for Special Engineered Fittings, Appurtenances or Valves for Use in Poly (Vinyl Chloride)(PVC) or Chlorinated Poly (Vinyl Chloride)(CPVC) Systems
F 1986-06	Standard Specification for Multilayer Pipe Type 2, Compression Fittings, and Compression Joints for Hot and Cold Drinking-Water Systems
F 1987-06	Standard Specification for Multilayer Pipe Type 2, Compression Fittings, and Compression Joints for Hydronic Heating Systems
F 2021-06	Standard Guide for Design and Installation of Plastic Siphonic Roof Drainage Systems
F 2160-08	Standard Specification for Solid Wall High Density Polyethylene (HDPE) Conduit Based on Controlled Outside Diameter (OD)
F 2206-02	Standard Specification for Fabricated Fittings of Butt-Fused Polyethylene (PE) Plastic Pipe, Fittings, Sheet Stock, Plate Stock, or Block Stock
F 2262-05	Standard Specification for Crosslinked Polyethylene/Aluminum/ Crosslinked Polyethylene Tubing OD Controlled SDR9
F 2306-07	Standard Specification for 12 to 60 in. [300 to 1500 mm] Annular Corrugated Profile-Wall Polyethylene (PE) Pipe and Fittings for Gravity-Flow Storm Sewer and Subsurface Drainage Applications
F 2435-07	Standard Specification for Steel Reinforced Polyethylene (PE) Corrugated Pipe
F 2562-08	Specification for Steel Reinforced Thermoplastic Ribbed Pipe and Fittings for Non-Pressure Drainage and Sewerage
F 2619-07	Standard Specification for High-Density Polyethylene (PE) Line Pipe
F 2623-08	Standard Specification for Polyethylene of Raised Temperature (PE-RT) SDR 9 Tubing
F 2648-07	Standard Specification for 2 to 60 inch [50 to 1500 mm] Annular Corrugated Profile Wall Polyethylene (PE) Pipe and Fittings for Land Drainage
F 2649-08a	Standard Specification for Corrugated High Density Polyethylene (HDPE) Grease Interceptor Tanks
F 2720-08	Standard Specification for Glass Fiber Reinforced Polyethylene (PE-GF) Spiral Wound Large Diameter Pipe

When the original specification was published, the manufacturing equipment and methods in producing plastics pipe and fittings was dramatically different than today. The original standard was based upon use of colored compounds in the production of pipe and fittings. The predominant means today in the manufacture of pipe and fabricated fittings is use of a “salt and

pepper” blend of a natural compound and colored master batch or concentrate. Fittings produced through the use of injection molding continue to mainly use colored compounds. Additionally, as in any progressive growth industry, materials today are significantly improved in ductility, slow crack growth properties, and other performance properties over the early generation materials. How this standard has transformed as the industry has evolved is the topic of this paper. Its intent is to provide the reader with a document that captures all the significant revisions to a standard specification that has become the foundation to identify materials for development of polyethylene piping application standards.

Chronological Scope Revisions of ASTM D 3350

The original scope of ASTM D 3350-74 was that this specification covers the identification of polyethylene plastic pipe and fittings materials according to a cell classification system. It is not the function of this specification to provide specific engineering data for design purposes.

In 1984, a section 1.2 was added to the scope, which addressed the term materials. It stated that the term, materials, as used in this specification means polyethylene (PE) resins with the added compounding ingredients. These are often called PE compounds or PE plastics. The terms materials, compounds, and plastics are synonymous in this specification. In 1993, this section 1.2 was removed and the terms materials and PE compounds were placed in the terminology section 3.2, Descriptions of Terms Specific to This Standard.

In 1996, the scope was expanded to incorporate changes that clarified the intent of the specification for the benefit of regulatory agencies and add a statement on recycle materials. No further modifications have been made to the standard's scope. Thus, the scope of ASTM D3350-08 is as follows:

1. Scope.

- 1.1 This specification covers the identification of polyethylene plastic pipe and fittings materials in accordance with a cell classification system. It is not the function of this specification to provide specific engineering data for design purposes, to specify manufacturing tolerances, or to determine suitability for use for a specific application.
- 1.2 Polyethylene plastic materials, being thermoplastic, are reprocessable and recyclable. This specification allows for the use of those polyethylene materials, provided that all specific requirements of this specification are met.

Chronological Revisions to ASTM D 3350

The following are significant revisions to the standard as a result of deletion of referenced ASTM standards, development of new ASTM test methods, modification to industry manufacturing practices, and improvement in polyethylene materials:

TABLE 2—*Color and UV stabilizer in ASTM D 3350-08a.*

Code Letter	Color and UV Stabilizer
A	Natural
B	Colored
C	Black with 2% minimum carbon black
D	Natural with UV stabilizer
E	Colored with UV stabilizer

TABLE 3—*PENT (ASTM F 1473) cell class in ASTM D 3350-98.*

PENT	1	2	3	4	5	6
Hours	0.1	1	3	10	30	100

TABLE 4—*Density cell class (pre-1999 and D 3350-99).*

Density	1	2	3	4
Pre-1999	0.910–0.925	0.926–0.940	0.941–0.955	>0.955
D 3350-99	0.925 or lower	>0.925–0.940	>0.940–0.955	>0.955

- 1984—Added Section 6.2 *Color and UV Stabilizer*. The color and UV stabilization shall be indicated at the end of the cell classification by means of a letter designation in accordance with the code in Table 2.
- 1996—Added an environmental stress-crack resistance (ESCR) (ASTM D 1693) cell class 4 representing test condition C, F20 > 600 h.
- 1998—
 1. Eliminated reference to ASTM D 1248 as a result of standard name change from Specification for Polyethylene Plastics Molding and Extrusion Materials to Specification for Polyethylene Plastics Extrusion Materials for Wire and Cable.
 2. Deleted cell class 6 for melt index; cell class 6 represented materials that have a melt index less than 0.15 and a flow rate not greater than 0.30 g/10 min when tested at condition 310/12.5.
 3. Added cell class 0 for unspecified and cell class 7 for specified values.
 4. Added PENT (ASTM F 1473) as an alternative means to slow crack growth resistance property with the cell class values in Table 3.

TABLE 5—*Density cell class (pre-2005 and D 3350-05).*

Density	1	2	3	4	5
Pre-2005	0.925 or lower	>0.925–0.940	>0.940–0.955	>0.955	
D 3350-05	0.925 or lower	>0.925–0.940	>0.940–0.947	>0.947–0.955	>0.955

TABLE 6—Primary properties^a—cell classification limits.

Property	Test Method	0	1	2	3	4	5	6	7	8
1. Density, g/cm ³	D 1505	Unspecified	0.925 or lower	>0.925–0.940	>0.940–0.947	>0.947–0.955	>0.955	...	Specify value	
2. Melt index	D 1238	Unspecified	>1.0	1.0–0.4	<0.4–0.15	<0.15	b		Specify value	
3. Flexural modulus, MPa (psi)	D 790	Unspecified	<138 (<20000)	138–<276 (20000–<40000)	276–<552 (40000–80000)	552–<758 (80000–110000)	758–<1103 (110000–<160000)	>1103 (>160000)	Specify value	
4. Tensile strength at yield, MPa (psi)	D 638	Unspecified	<15 (<2200)	15–<18 (2200–<2600)	18–<21 (2600–<3000)	21–<24 (3000–<3500)	24–<28 (3500–<4000)	>28 (>4000)	Specify value	
5. Slow crack growth resistance										
I. ESCR	D 1693	Unspecified								
a. Test condition (100% Igepal.)			A	B	C	C	...	...	...	Specify value
b. Test duration, h			48	24	192	600				
c. Failure, max, %			50	50	20	20				
II. PENT, h	F 1473									
Molded plaque, 80°C, 2.4 MPa		Unspecified	...	...	...	10	30	100	500	Specify value
Notch depth, F 1473, Table 1		Unspecified								
6. Hydrostatic strength classification										
I. Hydrostatic design basis, MPa (psi), (23°C)	D 2837	NPR ^c	5.52 (800)	6.89 (1000)	8.62 (1250)	11.03 (1600)	...	...		
II. Minimum required strength, MPa (psi), (20°C)	ISO 12162	...	...	...	...	...	8 (1160)	10 (1450)		

^aCompliance with physical properties in accordance with Section 8 is required including requirements for cell classification, color, and ultraviolet (UV) stabilizer, thermal stability, brittleness temperature, density, tensile strength at yield, and elongation at break.

^bRefer to 10.1.4.1.

^cNPR = not pressure rated.

- 1999—Clarified density cell classes as in Table 4.
- 2001—
 1. Added minimum required strength values of 8 and 10 MPa for hydrostatic strength classification as cell classes 5 and 6, respectively.
 2. Added ISO 9080 and ISO 12162 to the referenced documents.
- 2005—
 1. Modified PENT cell classes.
 - a. Removed PENT cell classes 1, 2, and 3 representing 0.1, 1, and 3 h, respectively.
 - b. Added PENT cell class 7 representing 500 h.
 2. Modified density cell class as in Table 5.

ASTM D 3350-08a Cell Classification Table 6

Table 6 sets forth the cell classification and related footnotes from the most recently published D 3350 standard at the time of this symposium. The most recent modification for D 3350-08a was to add a clarifying footnote c to Table 6 for defining the abbreviation NPR as not pressure rated.

Although the D 3350 standard is referenced in many application standards developed in the F17 Committee on Plastic Piping Systems, the jurisdiction of this standard is under the D20 Committee on Plastics and is the direct responsibility of Subcommittee D20.15 on Thermoplastic Materials. As a result, numerous projects have been initiated in D20.15.01 Polyethylene Section to revise and update the standard to industry needs.

Current D20.15.01 Polyethylene Section Projects Involving D 3350

- Project X15-955—Global review of standard to identify areas in standard that need modifications, additions, or clarifications.
- Project X15-1019—Addresses the use of extensometer or grip separation in determining the tensile and elongation properties.
- Project X15-1020—Provides definition or clarification to test sample requirements when conducting the testing to the properties in the cell classification.
- Project X15-1069—Addresses the use of flow rate to represent the melt index of <0.15 g/10 min, cell class 4.

Summary and Conclusion

Additions and revisions to ASTM D 3350 Standard Specification for Polyethylene Plastics Pipe and Fittings Materials have occurred over the last 35 years to address the industry environment on use of improved materials and manufacturing options. With over 40 ASTM standards referencing this material standard specification, the importance of this standard for identification of polyethylene materials in the plastics piping industry cannot be overestimated.

DESIGN

Michael Pluimer¹

A Service Life Assessment of Corrugated HDPE Drainage Pipe*

ABSTRACT: The service life of our nation's infrastructure has become a topic of increasing focus in recent years. While above-ground infrastructure deterioration is readily apparent, the infrastructure beneath the surface can be much worse and yet is often largely ignored until it is too late. Since culverts and other buried structures typically are not inspected as frequently as above-ground structures such as bridges, the proper assessment and determination of their service life becomes very important. The design service life of corrugated high density polyethylene (HDPE) drainage pipe has been a subject of considerable research over the past several years. This paper presents a method for determination of long-term service life of corrugated HDPE pipe by utilizing some of the current widely accepted methods employed by the smooth-walled plastic pipe industry while modifying them somewhat to take into account the unique geometry and installation conditions of buried corrugated pipe. The resulting data when tested in accordance to this protocol indicate a service life in excess of 100 years.

KEYWORDS: service life, HDPE, corrugated, pipe, 100 years, culvert, polyethylene, plastic

Introduction

The process for long-term service life prediction is three-fold: First, the structural failure modes of the pipe system must be identified. Second, the anticipated service conditions of the drainage pipe must be assessed, including factors such as environmental conditions, soil and traffic loads, and the

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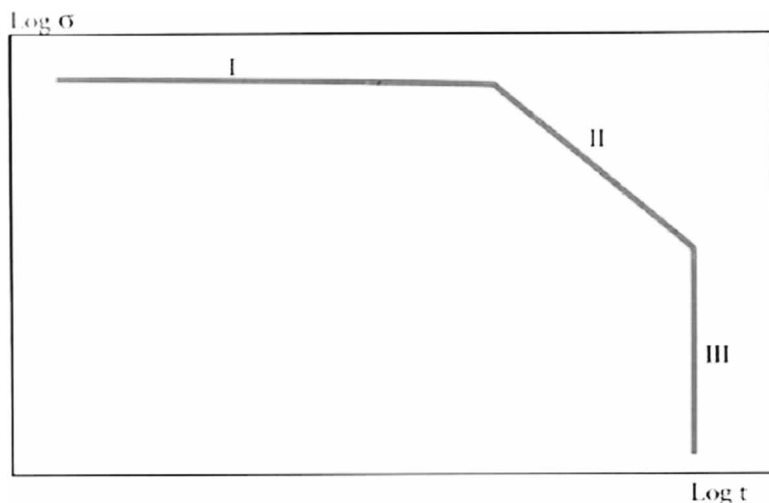


FIG. 1—Relationship between stress and time to rupture for polyethylene [6].

resulting long-term stresses and strains evident in the pipe. Third, the capacity of the material, the manufactured pipe, and the overall system must be assessed.

For corrugated high density polyethylene (HDPE) drainage pipe, the primary mechanisms of material failure are slow crack growth and oxidation or chemical failure. The service conditions of the pipe will vary by geographic location, based on temperature and soil and traffic loads. Recent research performed by Dr. Timothy McGrath for the Florida Department of Transportation (DOT) [1] details the limiting stress conditions of buried corrugated pipe, which will be used to predict the service life of pipe in this paper. After identifying the failure modes and long-term stresses on the pipe, the capacity of the material and the pipe system to resist failure is assessed. Some recently proposed methods by Dr. Grace Hsuan for the Florida DOT to ensure long-term material resistance to these failure modes are presented in this paper [1]. Florida DOT has a 100-year service life requirement for pipe, which will be the focus of this paper.

In addition to the pipe material, one also must consider the joints, gaskets, and installed pipe system in assessing service life. Many of these considerations have not been adequately addressed and are a suggestion for future research.

Identifying the Failure Modes of Corrugated HDPE Pipe

In order to adequately assess the service life of any structure, one must first identify the modes of failure. For the purposes of this paper, we will focus on slow crack growth and oxidation as the primary modes of failure for polyethylene pipe materials.

It is well-known that HDPE exhibits 3 modes of failure, depending on the stress level and chemical resistance evident in the material, as shown in Fig. 1.

Stage I failures are ductile in nature and occur at very high stress levels (much greater stresses than evident in gravity flow or low-pressure drainage pipe). Stage I failures are of little concern for corrugated HDPE pipe used for gravity flow drainage applications. Stage II failures are brittle types of fractures and occur at moderate stress levels. This is one of the primary failure modes for corrugated HDPE pipe and has been a main motive for the evolution of material performance requirements over the past decade. This mode of failure is associated with slow crack growth, a phenomenon that occurs in polyethylene pipe that is typified by crack propagation at low stress levels. Current materials for corrugated HDPE pipe include an notched constant ligament stress test requirement per ASTM F2136 [2], to ensure resistance to slow crack growth and stage II failures. Stage III failures occur as a result of chemical degradation of the material, and the steep curve associated with stage III failures indicates that the material no longer has the capacity to withstand any load. In order to prevent the onset of stage III failure in the pipe's anticipated service life, antioxidants are added to the material formulation.

To establish the 100-year service life for corrugated HDPE drainage pipe, one must ensure that the stress level in the pipe is lower than the long-term tensile strength of the pipe, thus avoiding stage I failure potential, and that the service stress and service conditions will not result in stage II or III failures prior to its anticipated service life.

Additionally, one must determine which failures are critical to the long-term performance of the pipe. Some may argue that any crack in the pipe should be considered a failure. However, certain elements of the profile (e.g., the liner of corrugated smooth-wall pipe) contribute minimally to the overall structural performance of the installed conduit, and an argument could be made that a failure to such components should not be considered critical, as the structural integrity of the pipe system as a whole will remain intact. Also, it should be noted that reasonable deflections (7.5 % or less) and waviness of the interior liner are not and should not be considered as failures since they do not affect the long-term structural performance of the pipe, provided that the stresses remain below the levels specified in this paper.

Evaluation of the Service Conditions of Buried Corrugated HDPE Pipe

Determination of the service conditions is the second step to evaluating the long-term performance of corrugated HDPE pipe. Flexible pipes rely on the soil for support; so, the pipe-soil system must be considered when determining the service conditions. To determine the maximum demand placed on the pipe material, one needs to address the various loading conditions.

Typically, the total stress in the pipe is a combination of the bending stress due to deflection and the hoop compression stress due to soil pressure. This may be represented by

$$\sigma = \frac{P}{A} \pm \frac{Mc}{I} \quad (1)$$

where:

σ = 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., and

I = moment of inertia of pipe wall, in.⁴/in.

Hoop stress is always compressive and increases with increasing hoop thrust (e.g., increased cover height). Polyethylene materials are much more resistant to compressive failure than tensile failure. Thus, the goal is to determine the maximum tensile stress in the pipe wall. Since the hoop compression stress is compressive, it will reduce the overall tensile stress in the pipe wall. To determine the maximum tensile stress, the hoop compression stress should be minimized while the bending stress maximized. The worst-case condition is thus shown to be shallow installations (low hoop thrust) with high deflections (large bending stress) [1]. 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.

Based on this installation condition, McGrath and Hsuan utilized the finite element analysis along with theoretical calculations using the AASHTO design method to determine that the strain limit for circumferential effects is 1.5 % [1]. This assumes a total vertical deflection of 5 % with minimal thrust. Using a long-term modulus of elasticity of 20 000 psi, the maximum long-term stress induced on the pipe wall is 300 psi. Applying an additional factor of safety of 1.5 results in stress and strain limits of 450 psi and 2.25 %, respectively (McGrath recommends slightly more conservative values of 500 psi stress and 2.5 % strain in his report to the Florida DOT).

Some states and municipalities limit vertical deflection of corrugated HDPE pipe to 7.5 % rather than 5 %. A limiting deflection of 7.5 % would result in a strain limit of 2.25 % and a long-term stress of 450 psi. Applying a factor of safety of 1.5 results in a strength limit of approximately 675 psi based on an installation with 7.5 % deflection.

The above calculations only take into account the circumferential stresses in the pipe. In order to adequately address the total state of stress in the pipe, the longitudinal stresses must be addressed as well. Citing two papers from Dr. Ian Moore, McGrath and Hsuan show that the longitudinal stresses in the pipe are on the same order of magnitude as the circumferential stresses or approximately 300 psi, equivalent to 1.5 % strain [1].

To summarize, the work by McGrath for the Florida DOT shows that the peak tensile stress in the pipe occurs at shallow installations and high deflections. Limiting the vertical deflection to 5 % results in a peak long-term stress of around 300 psi, equivalent to a strain of 1.5 %. If the deflection is limited to 7.5 %, the peak long-term stress in the pipe is around 450 psi, equivalent to a strain of 2.3 %. It should be emphasized that these peak stress levels refer to the general stress in the pipe and are not associated with areas of stress concentration [1].

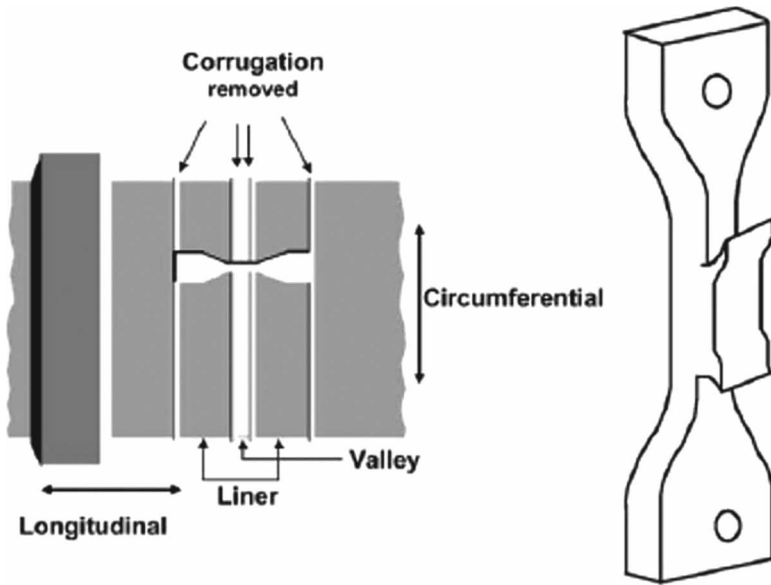


FIG. 2—Diagram of junction specimen used for evaluating pipe's long-term resistance to stage II failures [1].

In order to ensure the 100-year service life, the capacity of the material must be able to withstand this demand, which is the subject of the next section of this paper.

Evaluation of the Long-Term Material Performance of Corrugated HDPE Pipe

The previous part of the paper established the maximum factored stress in the pipe to be 450 psi for 5 % deflection and 675 psi for 7.5 % deflection. These are the critical stress levels that will be evaluated for the service conditions of the pipe.

Resistance to Stress Cracking and Stage II Failure

To evaluate the material's resistance to stage II failure, Hsuan proposed some new test methods to the Florida DOT. One of the primary tests was the evaluation of the junction between the corrugation and the liner, as shown in Fig. 2 [1]. This test specimen was chosen based on the prior work by Hsuan and McGrath in NCHRP Report 429 [3], where slow crack growth failures were noted on field samples of corrugated HDPE pipe at the liner–corrugation junction, as shown in Fig. 3. By the nature of the geometry of this junction, it will act as a stress concentration point, where SCG failure is most likely to occur.

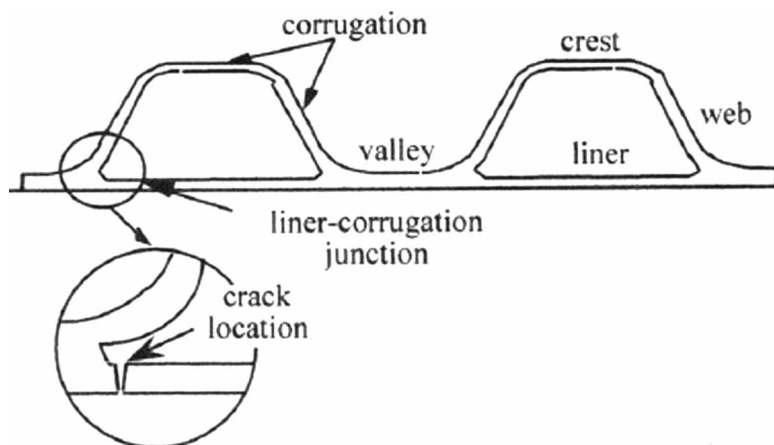


FIG. 3—Location of liner-corrugation junction where cracks were observed in NCHRP Report 429 [3].

This proposed junction test consists of a tensile load applied to the test specimen while immersed in a water bath.

It should again be noted, as stated earlier, that one can question whether or not cracking at this junction should be considered a critical failure, since the liner is a generally non-structural member of the pipe profile (it is intended primarily for hydraulic purposes), and the pipe will still continue to function as a structure even if the liner exhibits circumferential cracking. However, Florida DOT took the position that any cracking—whether in the liner or a structural member of the profile—would be considered a failure by their standards. As such, the Florida DOT protocol was developed around failures at the junction.

To determine the 100-year service life, the rate process method (RPM) can be utilized, which 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. RPM is widely used for the long-term service life prediction of HDPE pressure pipe materials and is part of ASTM D2837 [4]. This same method can be applied to corrugated HDPE pipe to determine the long-term service life.

RPM consists of an equation that relates failure time and stress as a function of temperature. The general equation is shown below in

$$\log t = A + \frac{B}{T} + \frac{C \log \sigma}{T} \quad (2)$$

where:

t = failure time, h,

σ = applied stress,

T = test temperature, K, and

A, B, C = constants based on material and test conditions.

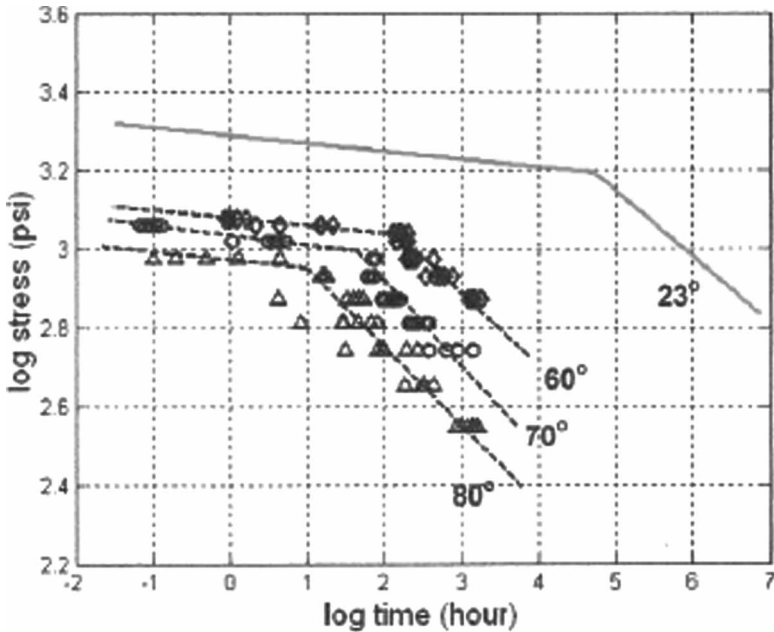


FIG. 4—An example of a predicted 23°C curve on a pipe junction in water [1].

A, B, and C in Eq. (2) are dependent on the material and test conditions and are determined by performing junction stress tests to failure at various load and temperature conditions. Once the three constants are determined, the equation may be used to predict time to failure at other stress and temperature conditions where the materials properties are known to remain constant.

For the Florida DOT 100-year service life protocol, Dr. Hsuan requires junction tests to be performed at the following conditions to determine the 3 constants:

- (1) 80°C, 650 psi stress,
- (2) 80°C, 450 psi stress, and
- (3) 70°C, 650 psi stress

Once the 3 constants are determined, Eq (2) is used to determine if the time to failure at the required service conditions (500 psi stress and 23°C for the state of FL) is greater than 100 years. An example of the extrapolated data is shown in Fig. 4 for one of the pipes used in the initial evaluation. The data clearly demonstrate that the RPM methodology is applicable to the evaluation of these types of specimens to predict slow crack growth failures. This example shows an estimated service life of well over 100 years for either the 500 psi condition associated with a 5 % deflection or the 675 psi condition associated with a 7.5% deflection (in fact, in this example, a service condition of 1000 psi stress and 23°C will give a 100-year design life). It is the recommendation of the Plastics Pipe Institute to limit deflections to 7.5 %, and initial tests show that the long-term performance of the material at the resulting factored stress level of 675 psi is well over 100 years.

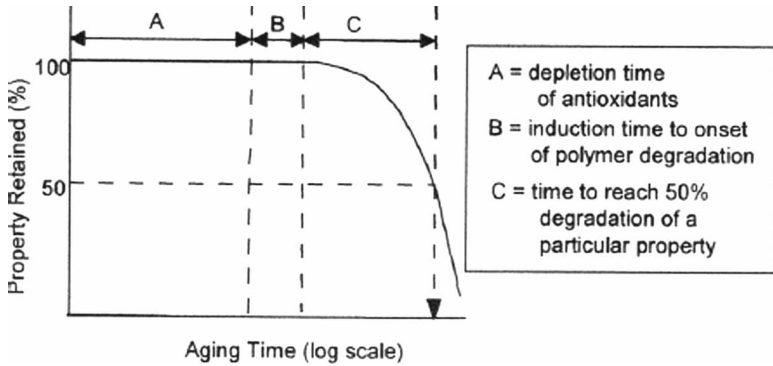


FIG. 5—Three conceptual stages for oxidation of HDPE [5].

While the RPM methodology provides for an adequate assessment of the service life of the pipe, it has some inherent practical limitations if we are simply trying to ensure that the materials meet a minimum service life requirement (100 years in the case of FL). Namely, it requires specimen failures in order to determine the constants and complete the extrapolations. Using this method as a qualification test for different pipe materials and profiles yields some concerns, particularly for very good materials that exhibit extremely long failure times. As such, Florida DOT developed single-point minimum test criteria to ensure that high-performing materials are not penalized when testing to a 100-year minimum service life requirement. Specifically, the tests may be terminated after a minimum of 110 h for the 80°C/650 psi condition, 430 h for the 80°C/450 psi condition, and 500 h for the 70°C 650 psi condition. These minimum failure times are based on Popelar shift factors applied to typical corrugated HDPE pipe materials with a 95 % confidence interval.

Resistance to Oxidation and Stage III Failure

To ensure that the stage III failures do not occur in the pipe prior to its designed service life, further testing is needed. As discussed above, stage III failures are prevented by adding antioxidants to the material formulation. The type and quantity of antioxidant play an important role in protecting the resin from oxidative degradation. Typically, as long as an antioxidant is present in the material, the polyethylene resin will be adequately protected from oxidation. 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 a stage III failure in this time period.

Hsuan and Koerner show that the overall oxidation mechanisms can be divided into three stages, shown in Fig. 5 [5].

It is critical that the onset of the stage III failures is beyond the anticipated service life of the pipe (100 years in this case). As shown in Fig. 6, a lack of antioxidants or the wrong type of antioxidant can shift the stage III failure curve to the left. If shifted far enough left, the service life of the pipe can be drastically

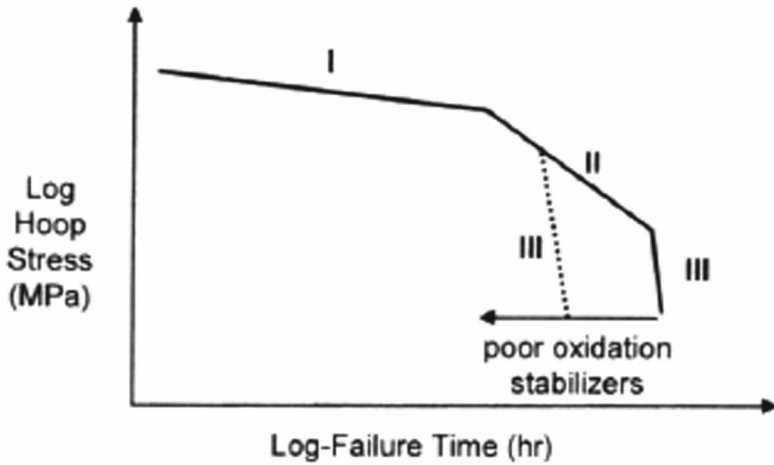


FIG. 6—A lack of antioxidants will shift stage III failures to the left, potentially limiting the service life [1].

shortened. However, if sufficient antioxidants are present, the onset of oxidation and stage III failure can be pushed out to well over 500 years [6].

There are two primary tests available to determine the antioxidant activity in a polyethylene formulation: (1) The induction temperature (IT) test (also known as thermal stability) and (2) The oxidation induction time (OIT) test. The IT test is performed by heating a test specimen at a constant rate and recording the temperature at which oxidation initiates. The OIT test is conducted by measuring the time to achieve oxidation at a given test temperature. Dr. Hsuan opted for the OIT test to evaluate the antioxidant activity of corrugated HDPE pipe for the Florida DOT protocol, which is consistent with international standards.

In order to adequately determine the sufficiency of the antioxidant to preventing the stage III failures prior to the pipe's intended service life, Hsuan proposed a minimum initial OIT value as well as a minimum retained value after incubation in an accelerated test environment. The minimum initial OIT value is 25 min and is based on data from HDPE geomembranes in a soil/water environment. The retained OIT value is 3 min and is to be measured after specimen incubation in a water bath at 80°C for 195 days, while under a constant tensile stress of 250 psi to simulate field conditions. This incubation period and temperature was selected based on the Arrhenius equation and the desired service life of 100 years in a 23°C environment. This work is detailed in the Florida DOT protocol [1].

To further ensure that no degradation of the material properties has occurred, Florida DOT requires a physical property test at the end of the incubation period in addition to the 3-min OIT requirement. Specifically, they require a melt index test to ensure the polymer has not degraded. Melt index is a good indicator of the molecular weight of the polymer and, as such, can be used to assess the material properties after immersion to ensure the antioxidant served

its purpose in protecting the polymer from chemical attack. Florida DOT requires a final melt index value after the 195 days of immersion of greater than 80 % and less than 120 % the initial value.

It should be noted that the proposed OIT requirements represent a very conservative approach to the service life estimation, since they require no onset of oxidation prior to the 100-year design service life. As shown in Fig. 5, there is still considerable time after the depletion of antioxidants before material property degradation will occur.

Conclusions

In order to establish the 100-year service life for corrugated HDPE pipe for drainage and low-pressure applications, the failure modes of the pipe must be identified, the installation conditions must be evaluated to determine the demand placed on the pipe, and the material properties must be evaluated to determine its capacity to meet the demand. Dr. Tim McGrath has determined that the maximum factored tensile stress in a pipe with a vertical deflection of 5 % is 500 psi corresponding to a strain of 2.5 % [1]. It can then be shown that the maximum tensile stress in a 7.5 % deflected pipe is 675 psi corresponding to a strain of 3.38 %. To determine the material's capability at meeting these demands, Dr. Hsuan has shown that by performing elevated temperature testing at multiple stresses and applying the RPM to forecast stage II performance at the design service temperature and stress level (500 psi for 5 % deflection limits and 675 psi for 7.5 % deflection limits), along with appropriate antioxidant performance tests to ensure that the stage III failures will not occur prior to 100 years [1], HDPE corrugated pipe can be evaluated for a 100-year service life.

Corrugated plastic pipe manufacturers have shown that they can meet the requirements of this extensive protocol, and some manufacturers have now passed all of the required tests, thereby obtaining a 100-year service life approval in the state of FL. This is a significant advancement for the drainage pipe industry. The testing required to comply with this protocol is extensive and has taken years, but it provides the end user with the technical confidence that corrugated HDPE pipe is a promising solution for our currently deteriorating infrastructure.

Suggestions for Further Work

As our current infrastructure continues to deteriorate, we need to be more critical of new structures that are being installed in our ground. The entire systems for all pipe materials should be more thoroughly evaluated to ensure adequate long-term performance. This should include an analysis of the joints and gaskets to ensure their long-term performance meets the expectations of the specifying agency. The system is only as good as its weakest link.

As materials continue to evolve for all pipe types, the long-term service performance of the materials should continue to be assessed.

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Designing Stormwater Chambers to Meet AASHTO Specifications

ABSTRACT: Thermoplastic stormwater chambers have been successfully used on many projects to meet current federal regulations controlling discharges from newly developed sites. These structures are commonly located under parking lots or roadways and subjected to vehicular loads on a regular basis. Given this application, it is logical to design these structures for loads specified in the AASHTO LRFD Specifications for Highway Bridges, which provide design procedures for thermoplastic culvert pipe for use under roadways. However, experience has shown that many drainage engineers are not familiar with the AASHTO Specifications and have difficulty evaluating product manufacturer's claims relative to AASHTO requirements. This paper presents the key aspects of the AASHTO specifications that should be applied to stormwater chamber design, such as determining applicable loads, load factors, design of profile sections, and consideration of time dependent properties. Also addressed are design matters for chambers not directly addressed by AASHTO. Of particular importance in developing unique structures is the validation of designs and chamber safety levels through full scale testing.

Introduction

Thermoplastic stormwater chambers have been successfully used on many projects to meet current federal regulations controlling discharges from newly developed sites. ASTM F2418-09 [1], Standard Specification for Polypropylene (PP) Corrugated Wall Stormwater Collection Chambers, has been developed as a product standard for polypropylene stormwater chambers, and ASTM F2787-09 [2], Standard Practice for Structural Design of Thermoplastic Corrugated

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Wall Stormwater Collection Chambers, has been developed as a design standard for thermoplastic stormwater chambers. The chambers considered in this paper are primarily arch shaped open-bottom chambers (Fig. 1); however, the principles of this paper apply to all thermoplastic stormwater structures. These structures are commonly located under parking lots or roadways and subjected to vehicular loads on a regular basis. Given this application, it is logical to design these structures for loads specified in the *AASHTO LRFD Bridge Design Specifications* [3], which provide safety levels, loads, and design procedures for thermoplastic culvert pipe for use under roadways, and, in fact, ASTM F2787-09 is based on the AASHTO procedures for designing thermoplastic culverts. Many drainage engineers are not familiar with the AASHTO specifications and have difficulty evaluating product manufacturer's claims relative to AASHTO requirements. This paper presents the key aspects of the AASHTO specifications that should be applied to stormwater chamber design, such as determining applicable loads and load factors (LFs), and design of profile sections considering time dependent properties and other design issues for chambers not addressed by AASHTO. Of particular importance in developing unique structures is the validation of designs and chamber safety levels through full scale testing. This paper discusses primarily the key parameters to be considered and the critical design concepts. Engineers should refer to the ASTM and AASHTO documents for specific details. The implementation of this design approach is demonstrated in Bass [4].

AASHTO Design Criteria

AASHTO provides detailed design specifications for thermoplastic pipe but not for stormwater chambers; thus, designers must identify the appropriate portion of the specifications to apply. For example, stormwater chambers are arch shaped compared to round pipes, which means that the distribution of moments and thrusts will be different and the soil bearing capacity at the base

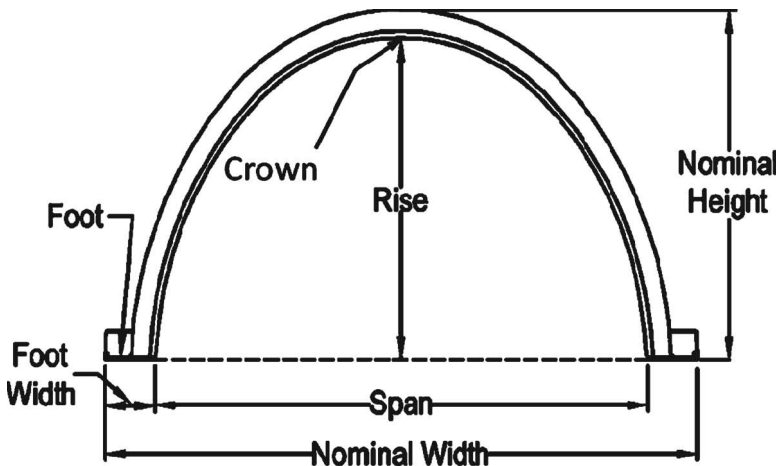


FIG. 1—Arch shaped stormwater chamber.

of legs and between chambers must be addressed. The key aspects of the AASHTO specifications for thermoplastic pipe that are applicable to stormwater chambers are loads and LFs, which establish the desired level of safety and the design procedures for profile wall sections, which establish the flexural and compressive capacity of the chamber wall. Although not normally applied to pipes, AASHTO provisions for determining bearing capacity can be applied to arch shaped stormwater chambers.

Loads

Loads on buried thermoplastic stormwater chambers and pipe consist primarily of earth loads and live loads. For live loads, the design of the most buried flexible pipe and stormwater chambers are controlled by the design truck (formerly called the HS-20 truck), which consists of a 32 000 lb axle equally distributed on two wheels, each with a contact area of 10 in. $\times$ 20 in. AASHTO increases this load by a factor of up to 1.33 to account for the impact and a multiple presence factor of 1.2 to account for overloaded trucks. Guidelines are provided for distributing load through fill. In stormwater chamber and thermoplastic pipe design, the beneficial effect of the pavement is normally ignored. This conservatively accounts for live loading during construction and for the softening of flexible pavements on warm days when loaded with parked vehicles.

In terms of magnitude, live loads are the dominant load for shallow installations, but they attenuate rapidly with increasing depth of cover. Live and earth loads are approximately the same magnitude at a depth of about 4 ft. At a depth of cover of 8 ft, AASHTO permits designers to drop consideration of live load. Live load pressure at the level of the chamber is computed as

$$\text{vehicle pressure} = \frac{\text{wheel load} * \text{LF} * I * m}{(T_W + H * \text{LLDF}) * (T_L + H * \text{LLDF})} \quad (1)$$

where:

vehicle pressure = live load pressure at the top of the chamber,

wheel load = design wheel load,

LF = load factor,

I = impact effect,

m = multiple presence factor to account for overloaded trucks,

T_W and T_L = length and width of the tire contact area, respectively,

H = depth of fill to the top of the chamber, and

LLDF = live load distribution factor, taken as 1.15 per AASHTO, which accounts for the increased distribution area with increasing depth of cover.

The equation is modified somewhat for deeper fills where load areas from multiple wheels overlap.

Equation (1) shows the factored design pressure at the top of the chamber for a 16 000 lb wheel load as 97, 34, and 17 psi at depths of cover of 6, 18, and 30 in., respectively. This rapid reduction of live load pressure with depth of fill allows validating chamber designs by reducing the depth of cover to increase the live load and avoiding the need to load trucks beyond their normal capacity.

Evaluating the cumulative magnitude of load conditions by looking solely at the magnitude of the load can be misleading due to the time dependent nature of thermoplastics. The long-term moduli of elasticity for polyethylene and polypropylene resins are typically on the order of only 20 % of the short-term modulus, and the modulus of elasticity is a key parameter in establishing the design strength of a chamber or pipe. Thus, if earth and live loads cause the same strain level in a short-term condition, the long-term response under earth load will be much larger than the live load response.

Load Factors and Limit States

The safety of buried thermoplastic stormwater chambers is established primarily through LFs. AASHTO requires that the thermoplastic pipe be designed with a LF of 1.95 on earth loads and 1.75 on live loads. The relative magnitude of these values is the reverse of typical values where the live LF is larger than the earth LF, but the significant point to make is that the factor of safety is of the order of 2.0 for both load types. Note that this factor is applied to long-term load conditions; thus, the expectation is that the factor of safety is fully preserved at the end of the design life of the chamber or pipe.

AASHTO design for thermoplastic pipe considers the strength limit states of tension, compression, combined flexure and compression, and global buckling. The controlling limit states are most often compression and combined compression and flexure. Earth loads at maximum depth of fill always control the long-term condition, while live loads are significant for short-term loads. Stormwater chambers, which are likely to be under parking lots, are often designed for both a moving truck load (vehicle passing over the chambers) and for a 1 week truck load (vehicle parked over the chambers for a period of 1 week). The 1 week load usually controls the design for shallow fills.

The most critical limit state for arch shaped stormwater chambers is compression capacity. Under most loading conditions, the chamber acts as a shell carrying the applied loads with compressive thrust and relatively small bending moments. Due to the relatively thin wall and the corrugated shape of the chamber, the key compression behavior is local buckling.

Designing for the Long-Term

Designing a stormwater chamber or a corrugated thermoplastic pipe for a 50-year design life results in a chamber having substantial overcapacity in most short-term tests. This will be discussed in more detail below in the section on in-ground chamber tests, but the overall concept is provided here.

Strains under short-term loads are of relatively low magnitudes; however, due to the drop in the apparent modulus of elasticity with time, the strains increase substantially during the chamber service life. Thus, to validate a design at a strength limit state, a short-term test must apply several times the design earth or live load to the chamber. In simplistic terms, test loads should be of the magnitude

$$\text{test load} = \text{design load} * \text{LF} * \text{effect of reduced modulus} \quad (2)$$

The means of developing this load are discussed in subsequent sections.

In computer modeling, earth loads and any other permanent conditions should be modeled using long-term properties for the thermoplastic materials. Live loads should be modeled for the appropriate load duration. Using 1-min duration to model moving live loads is appropriately conservative, and a modulus representing a sustained load of one week can be used to model a parked vehicle.

Bearing Capacity

Three bearing capacity issues need to be evaluated for an arch shaped stormwater chamber.

- (1) The foot of the chamber bears directly on foundation stone.
- (2) The stone column between chambers carries the majority of the weight of vehicles and earth above the base of the chamber and bears on the foundation stone.
- (3) The entire stormwater chamber installation bears on the native soils.

Each of these modes should be addressed in design. Mode 1 is important to preserve the chamber volume as a bearing failure would result in the chamber foot settling into the foundation stone but does not put the chamber installation at risk as the chamber leg load would simply transfer to the stone column. Mode 2 is critical to provide stability to the entire chamber installation. Mode 3 is also critical but is a function of the native soil conditions, thus a responsibility of the site engineer. Low native soil bearing capacity can normally be addressed through the use of thicker foundation layers.

Structural Model for Chamber Design

The structural behavior of a buried arched shaped stormwater chamber involves complex soil-structure interaction. The soil around the chamber plays a key role and, in the case of earth loads, a primary role in supporting loads. This behavior is captured in finite element programs that can model both soil and chamber properties appropriately. CANDE [5] and ABAQUS [6] are two finite element programs commonly used for this purpose.

Load-Path: Earth Loads

Structural design for vertical loads must always address the load path, that is, how the weight of earth above a chamber passes through or around the chamber to the foundation layer. In the case of thermoplastic arch shaped chambers, much of the load is carried by the stone embedment, as shown in Fig. 2. The load is carried by the stone because of the low axial stiffness of the chambers. This is demonstrated by examining an installation of a chamber meeting the 30 × 51 classification of ASTM F2418-09, which has a 30 in. nominal height and a 51 in. nominal width. The axial stiffnesses of the stone and the chamber are presented in terms of *EA*, the modulus of elasticity times the cross-sectional area, in Table 1.

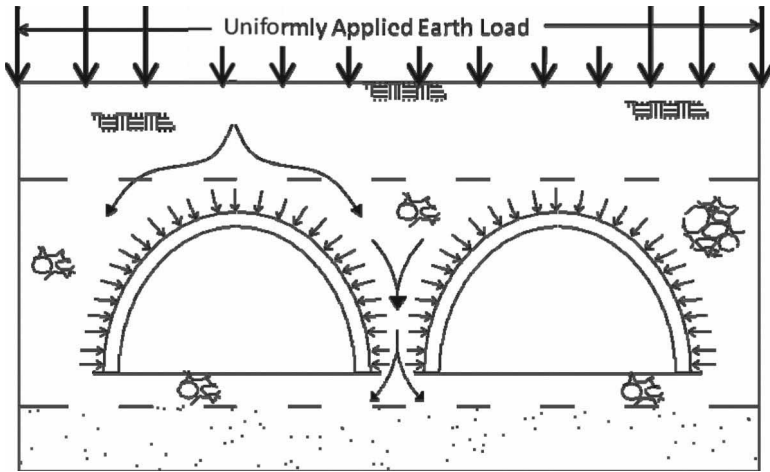


FIG. 2—Earth load flows around the chamber through the embedment.

Table 1 demonstrates that under short-term loading, the axial stiffness of the stone is modestly larger than the stormwater chamber, but in the long-term, due to the decrease in the apparent modulus of elasticity, the stone is much stiffer than the chamber. This means that over time, the chamber will compress more than the stone. As a result, the load initially carried by the chamber in the short-term arches to the stone, limiting the long-term compression strains in the chamber and allowing for efficient chamber designs. Note that compression strain increases over time even though the load is decreasing. For the 30×51 chamber noted above, the compression strains and thrust forces in the short- and long-terms are presented in Table 2.

Table 2 demonstrates the load shift from the chamber to the stone as a reduction in the thrust force carried in the wall. If the embedment material is not sufficiently stiff to accept the transferred load, the long-term strains would be much higher, and the possibility of chamber failure would increase.

Load Path: Live Loads

Since live loads are a short-term loading condition, the significant shift of thrust over time seen under earth load conditions does not occur. Also, because vehicle

TABLE 1—Vertical stiffness of embedment stone and stormwater chamber.

Parameter	Embedment Stone $E \sim 6000 \text{ psi}^a$	Stormwater Chamber	
		Short-Term $E \sim 150\,000 \text{ psi}$	Long-Term $E \sim 27\,000 \text{ psi}$
Axial stiffness, EA (lb/in.)	54 000	45 000	8100

^aAssumes a 9 in. average width of stone between rows of chambers.

TABLE 2—Factored compression strain and thrust force in the chamber with 8 ft of cover.

Strain (%)		Thrust Force (lb/in.)	
Short-Term	Long-Term	Short-Term	Long-Term
0.7	3.0	260	160

loads are moving over the top of the chambers in all directions, the live load effects will be variable. When a live load is located between rows of chambers, the load on the stone will push the chambers to the side, and when the live load is located over the chambers, the crown of the chamber will be pushed down. Arch shaped stormwater chambers require a minimum flexural capacity to resist the effects of moving live loads.

Chamber Capacity in Laboratory Tests

Stormwater chambers use thermoplastic resins that meet strength and modulus requirements based on design standards such as ASTM F2787-09 and AASHTO. The capacity of manufactured stormwater chambers can be evaluated through several laboratory tests.

Arch Stiffness Test

The arch stiffness test is similar to the parallel plate test conducted on thermoplastic pipe. The chamber legs are restrained against lateral motion, while the chamber is loaded vertically at the crown by a flat plate. Load and deformation are monitored during the test. The arch stiffness test (Fig. 3) determines two key parameters—stiffness and flexural strength. Stiffness is required to resist loads and deformation during the process of handling and installing chambers. Flexural strength is required to resist live loads during handling and installation.

Stiffness is represented by the arch stiffness constant (ASC) which is analogous to pipe stiffness for a round pipe. The ASC is the load to deform a unit length of chamber to 2 % deflection

$$ASC = P/L/2 \% \text{ (lb/ft/\%)} \tag{3}$$

where:

- P* = load on a test section at 2 % deflection and
- L* = length of the test section.

To evaluate chamber flexural strength, the arch stiffness test must continue with a constantly increasing load until the deflection (decrease in rise) exceeds 7 %. For current chamber designs, this deflection level produces approximately the limiting design strain for the chamber.

Stub Compression Test

The stub compression test was originally developed to validate the compression design procedure used by the AASHTO specifications for profile wall

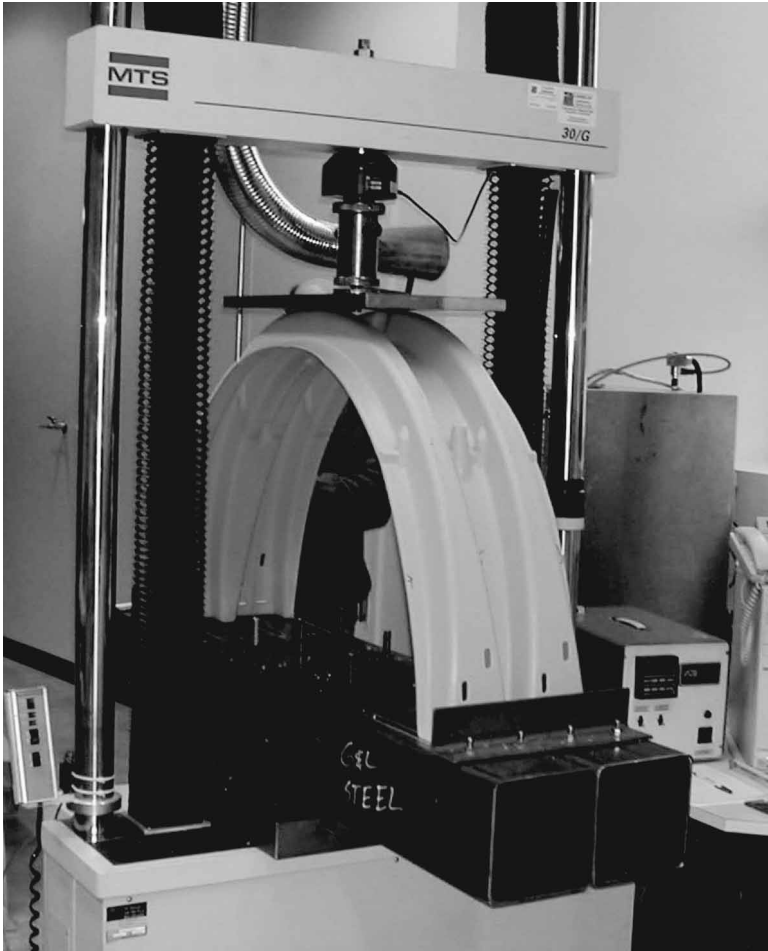


FIG. 3—Arch stiffness test.

thermoplastic culverts (McGrath and Sagan [7]). NCHRP Report 631 [8] has recommended that AASHTO adopt this test as a standard product qualification test for thermoplastic culverts, and this is currently in process. The stub compression test (Fig. 4) consists of compressing a circumferential portion of the pipe wall between two fixed parallel plates. The strain at peak load represents the compression capacity of the pipe wall under earth loads. The test is normally conducted on a wall section from the base of the leg where capacity under deep earth fill would be first reached.

Validation of Designs Through Field Tests

The behavior of stormwater chambers is sufficiently complex that it is desirable to validate the designs through full scale field testing. Some manufacturers

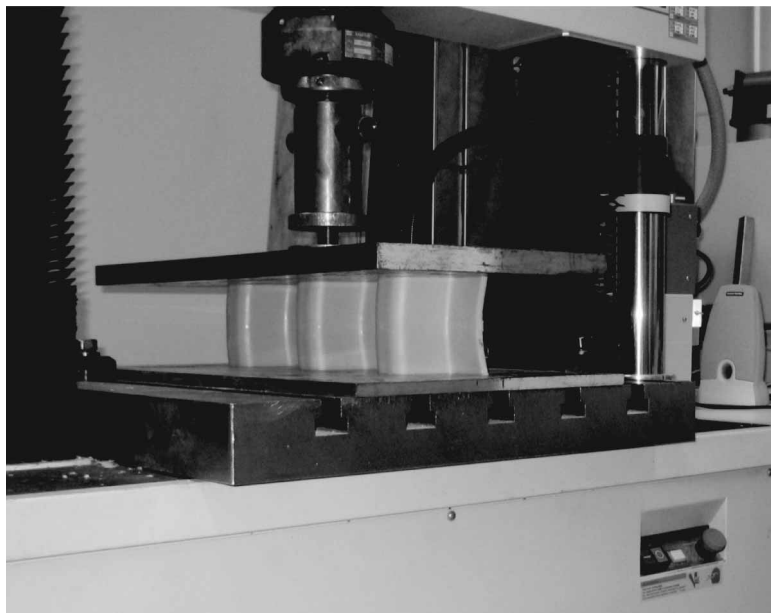


FIG. 4—*Stub compression test.*

validate chamber designs only through field testing. The key issue in conducting full scale field tests is to validate the design under the full factored loads and not just at the service load condition. This approach assures that the desired level of safety is provided in a chamber design.

Deep Fill Field Tests

Deep fill field tests demonstrate the stormwater chamber system capacity to carry vertical earth load. Under this loading, the compression force in the leg is typically the critical limit state. For a chamber designed for 8 ft of fill, the test depth of fill would need to be twice as great, or 16 ft, to demonstrate a factor of safety of about 2.0, but, as noted above, this only establishes safety for the short-term condition. Since the goal is to demonstrate the factor of safety of about 2.0 for the long-term, the modulus effect discussed above would need to be considered. An alternative to applying large depths of fill over a test chamber is to construct a test installation with soft backfill, such as clay. The soft backfill compresses under the earth load resulting in substantially more load on the chamber than would be experienced with a typical crushed stone backfill. In this way, the 30 × 51 chamber design was validated through a test at 11 ft of fill. The larger chamber size discussed in Bass *et al.* [4] was validated with stone backfill, close chamber spacing, and about 20 ft of fill. Another alternative to this type of test installation is soil box testing. Soil box tests allow chamber designs to be validated in a controlled environment with a reduced need to



FIG. 5—Live load testing of 30×51 chamber.

move large volumes of earth. Care must be taken in soil box tests to assure accurate modeling of field conditions.

Live Load Field Tests

Shallow burial tests demonstrate the stormwater chamber system capacity to carry vehicular traffic. Under this loading, flexure in the upper half of the chamber is typically the key limit state. Applying full factored loads to a storm chamber is difficult as the factored design loads are larger than can typically be accommodated with a test vehicle and such heavy vehicles cause severe rutting in the subgrade. Thus, it is a simpler task to conduct tests at shallower depths of fill than the design depth to produce the desired factored load effect. Figure 5 shows a live load test conducted on a 30 × 51 chamber. In this test, loads were applied through a front-end loader carrying up to four large concrete blocks. Tests were initially conducted at 2 ft of fill, after which the cover was reduced in 6 in. increments until chamber capacity was reached at a depth of 6 in. As noted above, the high load applied at 6 in. demonstrates a significant factor of safety if the design depth of fill is 18 in.

Conclusions

The design in accordance with AASHTO specifications provides stormwater chamber installations with the same safety level as AASHTO demands of thermoplastic culverts. This in turn assures users of installations of high quality and long service life. The key issues in designing in accordance with AASHTO and validating these designs through field tests have been presented and explained. Important criteria for chamber designs include the following:

- Design for factored earth and live loads to provide suitable safety levels for finished products,
- Proper consideration of long-term thermoplastic properties,
- Assuring a proper load path for earth and live loads above the chambers to the supporting foundation soil below the chambers,
- Consideration of compression stresses and local buckling effects through AASHTO specifications, and
- Validation of designs through laboratory and full scale testing.

A thorough analysis, design, and testing program that addresses these key issues will result in safe, long-lasting stormwater chamber installations. Current AASHTO specifications for thermoplastic pipe provide most of the guidance necessary for loads on chambers and evaluation of structural sections. Conformance with AASHTO standards should be a goal for stormwater chambers and other buried thermoplastic structures.

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Design and Performance of Plastic Drainage Pipes in Environmental Containment Facilities*

ABSTRACT: Plastic drainage pipes are an essential component of modern engineered barrier systems used in municipal and hazardous solid waste landfills. The design and performance of plastic pipes in these applications is significantly different than conventional municipal sewers and culverts. For example, the plastic pipe may be surrounded by very coarse gravel backfill which results in additional local bending stresses from the irregular loading and support for the pipe. These plastic pipes also experience local stress concentrations from the perforations required to collect fluid. Both of these factors are exacerbated by potentially large overburden pressures acting above the pipe. They must also perform their function while being exposed to chemicals in landfill leachate for long periods of time and at temperatures significantly larger than those for conventional sewers and culverts. The influence of coarse gravel backfill, large perforations, large overburden stresses, chemical exposure and elevated temperatures on the long-term performance of plastic drainage pipes is examined.

KEYWORDS: landfills, leachate collection pipes, perforations, antioxidant depletion

Introduction

Plastic drainage pipes are an essential component of modern engineered barrier systems used in waste containment facilities like municipal and hazardous solid waste landfills, Fig. 1(a). A network of perforated plastic pipes installed within a

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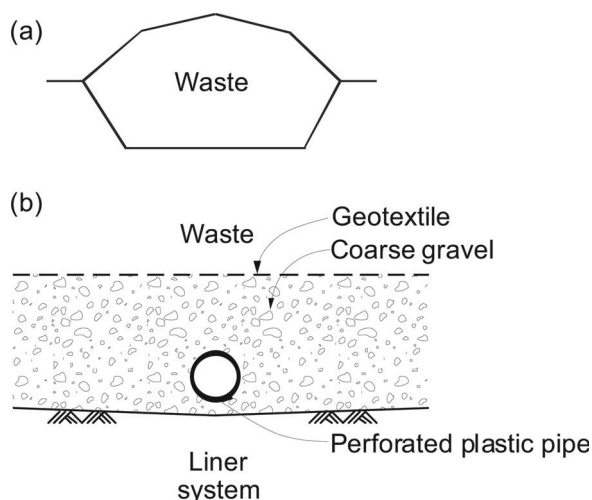


FIG. 1—(a) Solid waste landfill. (b) Laterally extensive leachate collection system with perforated plastic drainage pipe and coarse gravel backfill.

laterally extensive granular drainage blanket, Fig. 1(b), is an essential component of the leachate collection system that overlies the liner system. The liner system itself—comprised of native clayey soils or more commonly an engineered composite system with a geomembrane over compacted clay—serves to reduce the advective and diffusive migration of contaminants from the facility [1]. The purpose of the leachate collection system is to allow rapid transmission of contaminated fluid called leachate to removal points (i.e., sumps in manholes). As such, the leachate collection system provides a means to control the hydraulic head acting on the barrier, and thereby reduce the outward movement of contaminants by advective flow. It also provides an opportunity to remove contaminants which can shorten the contaminating lifespan of the facility (i.e., the period of time that the landfill has contaminant at levels that could have unacceptable impact if discharged into the environment [1]). Consequently, properly functioning drainage pipes are important for minimizing the environmental impact of the waste containment facility. The objective of this paper is to identify and discuss the factors influencing the design and performance of plastic drainage pipes in environmental containment facilities.

Unique Exposure Conditions and Design Considerations

The design and performance of plastic pipes in municipal and hazardous solid waste landfills is different than conventional municipal sewers and culverts. The principal differences arise because of clogging of the leachate collection system from suspended particles in the leachate and solid precipitates from chemical and biological interactions [1]. The available theoretical, experimental and field data (e.g., see [1]) overwhelmingly shows that leachate collection systems need to be designed to delay the onset of and minimize the impacts of

clogging. Clogging will greatly reduce the voids of the drainage material surrounding the pipes, plug the perforations in the pipe and possibly even the pipe itself unless the pipe is actively and frequently flushed. For example, observations at an exhumed landfill by Fleming et al. [2] revealed that after only 1 to 4 years of exposure to landfill leachate: (i) up to 80 % of the voids in the drainage gravel were clogged; (ii) 8-mm-diameter circular perforations in the pipe were mostly clogged; and (iii) there was significant accumulation of solid material in the pipe. By reducing the effectiveness of leachate collection, clogging may lead to a build up of leachate head acting on the liner system (especially if the rate of infiltration exceeds the reduced rate of leachate collection), and—with other factors being equal—may lead to greater contaminant transport from facility [1].

Clogging can be minimized by using coarse poorly-graded gravel in the leachate collection system as its large voids will take longer to clog than finer gravel or sand [1]. For example, landfill regulations in Ontario require the drainage gravel to have 85 % of the particles by mass not less than 37 mm, and 10 % of the particles by mass not less than 19 mm [3]. As a result, the plastic pipe may be surrounded by very coarse gravel backfill which results in local bending stresses from the irregular loading and support for the pipe [4]. Using perforations as large as possible in the pipe to permit leachate collection is desirable to minimize clogging of the hole itself, as a large circular hole would clog less rapidly than a smaller circular hole or a long and narrow slotted perforation of equivalent opening area. The plastic pipes will then also experience local stress concentrations from the perforations, depending on their shape, size, spacing and location around the pipe. Effects of both coarse gravel and perforations may be exacerbated by large overburden pressures acting above the pipe, which in some cases may come from burial beneath greater than 100 m of waste [5]. The pipes must also perform their function while being exposed to the chemicals in the leachate for long periods of time at temperatures much higher than conventional sewers [1].

The internal diameter of the pipe must be large enough to transmit the volume of leachate by gravity flow to a collection point. Equations for calculating flows in leachate collection pipes have been given by Giroud et al. [6]; however, the hydraulic capacity of these pipes is rarely the controlling factor as flow rates into the leachate collection system are normally small. Selection of the internal diameter is then based on maintenance requirements, such as being large enough to permit hydraulic flushing to clean any clogged material that has precipitated in the pipe and video camera inspection to verify the effectiveness of pipe flushing. Commonly encountered internal diameters may range from 150 to 300 mm.

High-density polyethylene (HDPE) pipes are normally used in landfills given their excellent chemical resistance to the wide range of chemicals found in landfill leachate. Conceivably, either thick plane-wall pipes (with solid prismatic geometry) or profile-wall pipes may be used. The design and performance of plane-wall HDPE pipes in landfills are the exclusive focus of this paper. Presumably the design and performance of profile-wall pipes would require similar considerations as detailed in the rest of this paper, but with additional consideration given to issues of local bending and local buckling in thin elements of profile-wall pipes (e.g., see [7]).

Adequate structural capacity of a plane-wall pipe is obtained by selecting an appropriate wall thickness of the pipe for the specific overburden pressures and backfill conditions such that pipe deflections and stresses are below allowable levels. For plane pipes, the pipe thickness is expressed using the dimension ratio (DR), which is equal to the external diameter of the pipe OD divided by the minimum wall thickness of the pipe t . Factors influencing the structural capacity and methods to select an appropriate pipe thickness are presented in the following sections; however, it is useful at this point to consider the possible implications of unacceptable pipe performance on the overall performance of the waste containment facility. A collapsed or even excessively deformed leachate collection pipe would inhibit pipe cleaning and thus would be more likely to clog; without remedial action, the build up of leachate head would likely result in greater contaminant transport from landfill (e.g., see [1] for details). In double liner systems (i.e., where a second leachate collection system—sometimes called a leak detection system—is placed beneath a composite geomembrane-clay liner) unacceptable performance of pipes in secondary leachate collection system may lead to excessive deflection and possibly rupture of the overlying liner.

Perforations

Perforations should be large enough to provide adequate flow into the pipe. Solutions exist to calculate the number and spacing of the perforations to provide adequate flow [8], although given the low flow rates in leachate collection systems, it is doubtful that limiting flow will govern design. Perforations should be small enough to minimize movement of fines from the backfill and so the maximum diameter of circular perforations should not exceed $[1/2]D_{85}$ of the backfill soil [9], where 85 % of the backfill soil material is finer than D_{85} by mass. They should be large enough to limit the extent of biologically induced clogging and also to maximize the effectiveness of cleaning (from pressurized hydraulic jets that pass through the pipe).

Perforations should also be small enough to limit increase of stress in the pipe. Brachman and Krushelnitzky [10] used three-dimensional finite element analysis to derive stress concentration factors that can be used with the elastic soil-pipe interaction solution of Hoeg [11] to provide the stresses in a perforated pipe under deep burial. Stress concentration factors were derived for the uniform normal, non-uniform normal and non-uniform shear boundary stresses acting on the pipe in Moore's formulation [12] of Hoeg's solution. The maximum stress around the perforation depends on the circumferential location of the perforation, and increase as the perforation approaches the crown, springline or invert. Consequently, from the perspective of stresses in the pipe, the best location for the perforations is at the quarter-points of the pipe as shown in Fig. 2 because the stresses are smallest near the shoulder and haunch locations of the pipe; conversely, the worst location is at the crown, springline or invert, since the stresses are the highest at these locations.

For circular perforations located at the quarter-point, stress concentrations depend on the thickness and diameter of the pipe, and the diameter of the perforation. They increase as the pipe becomes thinner and as the hole becomes

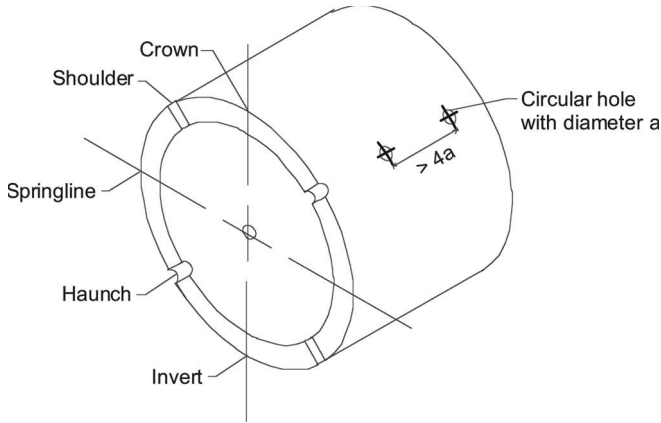


FIG. 2—Perforated drainage pipe with four circular holes located at the quarter points ($\pm 45^\circ$ from springlines).

bigger relative to the pipe diameter as shown in Fig. 3. The results in Fig. 3 also show that a stress concentration factor of 3 obtained from the simple hole in a plate solution can be conservatively used for most perforated pipes, except for very thin pipes ($DR > 35$) with large holes ($D/a < 12$).

The stress concentrations also depend on the axial and circumferential spacing between perforations. They increase as holes become more closely spaced in the axial direction. An axial centre-to-centre spacing of at least four times the perforation diameter between perforations is sufficient to avoid increases in the stress concentration factors.

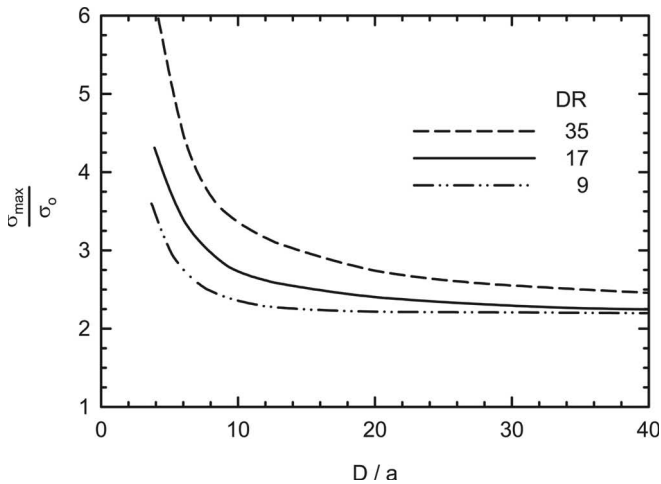


FIG. 3—Maximum stress (σ_{max}) around a circular perforation with diameter a in a pipe with mean diameter D and dimension ratio DR subjected to uniform normal boundary stress σ_0 . Modified from [10].

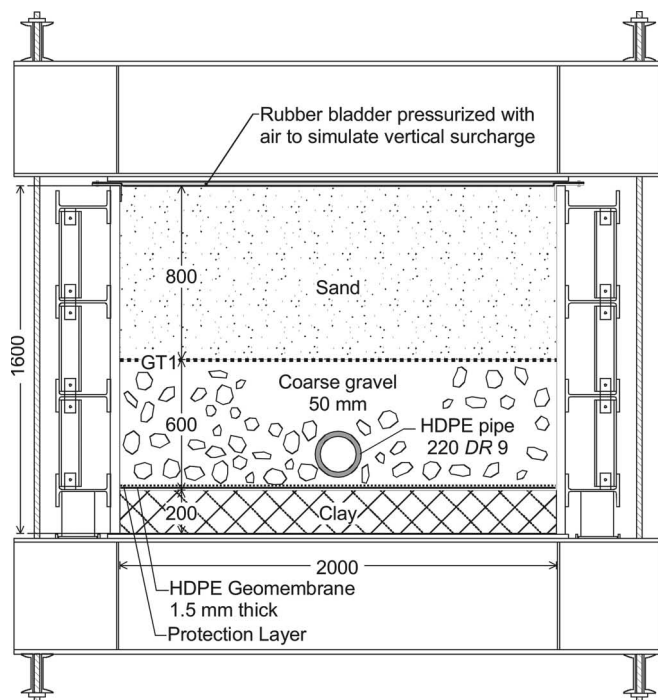


FIG. 4—Cross section through biaxial compression test cell showing simulated landfill conditions. Modified from [13]. Dimensions in mm.

Coarse Gravel Backfill

Because the gravel particles are relatively large with respect to the size of the pipe, coarse gravel both loads and supports the pipe at discrete points around the outside surface. Items of practical interest are then: what values of stiffness and coefficient of lateral earth pressure should be used for coarse gravel during pipe design and how to account for local effects from the coarse gravel on the structural performance of the pipe.

Large-scale laboratory tests have been conducted to measure the performance of plastic drainage pipes for landfill applications. Figure 4 shows a cross section through the test cell developed by Brachman et al. [13] to simulate the vertical and horizontal stresses in the drainage layer containing the pipe. The inside dimensions of the test cell are 2 m wide, 2 m long, and 1.6 m high. Vertical load was applied by a pressurized air bladder, which provided a uniform vertical pressure (up to 1000 kPa) across the top surface. Horizontal stresses corresponding to essentially zero lateral strain conditions develop in the soil because outward deflection of the sidewalls is limited. Boundary effects from friction between the soil materials and walls of the test cell were minimized using multiple polyethylene sheets lubricated with silicone grease along the sidewalls.

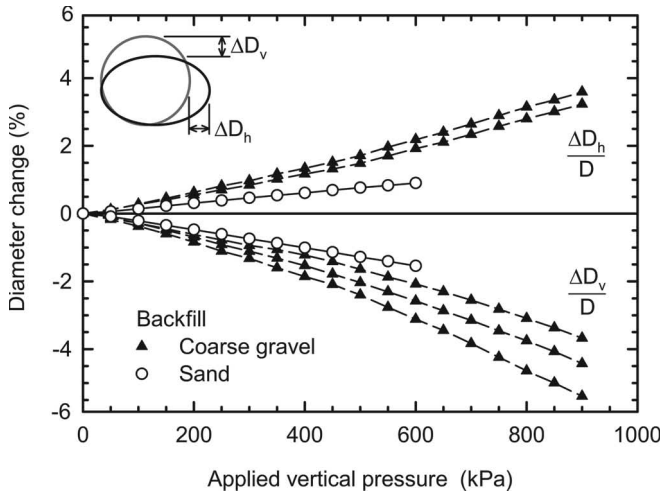


FIG. 5—Deflections of 220-mm-OD, DR-9, HDPE pipe from with sand and 50-mm coarse gravel backfill. Modified from [13].

One set of specific conditions of a 220-mm-OD, DR 9, HDPE leachate collection pipe in a gravel blanket was examined. The pipe was perforated with four circular holes with a diameter of 25 mm located at the shoulders and haunches of the pipe at centre-to-centre spacing of 150 mm along the length of the pipe. The pipe was placed within a 600-mm-deep blanket layer of nominal-50-mm, poorly-graded, coarse gravel (70 % finer than 51 mm and 8 % finer than 38 mm) that was dumped in-place to simulate the drainage layer. Results for a different backfill configuration (where the pipe was placed in a landfill trench rather than a laterally extensive drainage blanket) have been reported by Brachman and Krushelnitzky [14]. The gravel was crushed limestone and consequently the particles were angular. The pipe was placed with approximately 100 mm of gravel material between the pipe invert and the underlying 200 mm thick clay layer, included to simulate the effect of a compressible layer beneath the pipe. Poorly-graded, medium sand was used to fill the remainder of the test cell.

The deflections of the pipe were measured as the vertical pressure in the bladder was increased. Vertical ΔD_v and horizontal ΔD_h diameter changes (as a percentage of the initial average pipe diameter D) measured at three locations along the pipe are plotted in Fig. 5 for one test with a gravel blanket as shown in Fig. 4. For comparison, pipe deflections from a test conducted with the same pipe located in the middle of the test cell and backfilled with poorly graded medium-dense sand are also shown in Fig. 5. Burial in the gravel blanket resulted in larger pipe deflections predominantly because the coefficient of lateral earth pressure is lower for the gravel than the sand [13]. The pipe also experienced much greater variations in deflections in the gravel blanket compared with sand backfill because of the discrete support provided by the coarse gravel. The increase in pipe deflection induced by the coarse gravel needs to be accounted for during pipe design. The approach proposed here is first to

quantify the average response of the pipe induced by the coarse gravel backfill—assuming that there is some average response where the coarse gravel would behave more as an continuum—and then, second, use experimentally derived factors to account for local increases in pipe deflection from the variable loading and support provided by the coarse gravel.

Here, the elastic soil-pipe interaction solution of Hoeg [11] using the notation of Moore [12] is used to capture the average pipe response. The soil is treated as a uniform elastic material with Young's modulus E_s , Poisson ratio ν_s and coefficient of lateral earth pressure K ; likewise for the pipe with Young's modulus E_p and Poisson ratio ν_s . Values of pipe modulus for the strain rate of the large-scale laboratory tests were obtained from the viscoplastic model HDPE of Zhang and Moore [15,16]. A series of tests with 50-mm coarse gravel backfill and the simulated landfill conditions shown in Fig. 4 have been reported [17]. Secant values of soil modulus for the particular 50-mm coarse gravel tested when subject to stress conditions expected in a blanket layer can be inferred by comparing the average measured deflections from the laboratory tests with values calculated using Hoeg's solution [11]. Up to applied vertical pressures of 500 kPa, the secant modulus of the gravel was estimated to be $E_s = 30$ MPa, while for pressures between 500 to 900 kPa, the secant modulus was estimated to be 25 MPa. The small decrease in secant modulus of with increasing vertical pressure is attributed to modest softening arising from increased shear strains in the gravel. Poisson's ratio $\nu_s = 0.175$ and lateral earth pressure coefficient $K = 0.11$ were used to characterize the coarse gravel.

Based on statistical analysis of the measured variations from the laboratory tests the average diameter change of the pipe should be increased by a factor of 1.3 to account for variations induced by the coarse gravel [17]. This value was obtained by comparing the maximum to average measured response. A similar approach can be used to get stresses in the pipe. Stresses in the pipe were found to be more sensitive to local gravel effects. Maximum short-term compressive stresses were found to be 1.6 times the average response while maximum short-term tensile stresses at the invert were 2.0 times the average response [17]. These stresses were computed from measured strain values (using 2-mm long strain gauges) and a viscoplastic constitutive model (considering strain magnitude and rate affects) [16]. They are intended to account for variations induced by the coarse gravel loading and support. These gravel stiffness and magnification factors are applicable for the specific conditions, namely the particular 50-mm-gravel in a 0.6-m-deep blanket configuration and 220-mm, OD, DR9 HDPE pipes. The stiffness may be different for other backfill materials. The magnification factors may be even larger for thinner pipes, coarser gravel or pipes with smaller outside diameters, and for these cases can only be obtained from a series of large-scale laboratory tests.

Elevated Temperatures and Time

For plastic pipes in conventional sewer and culvert applications, long-term deflections are typically obtained by using a reduced pipe modulus anticipated for long-term conditions or by multiplying the calculated short term deflections

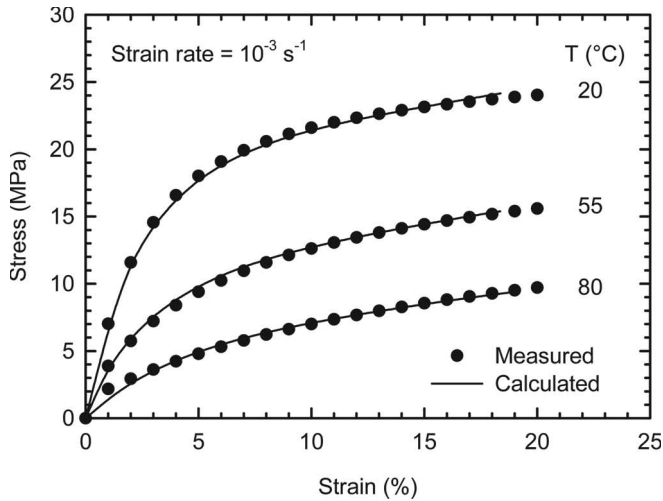


FIG. 6—Uniaxial compression test results on cylindrical specimens (25.4-mm-long, 12.7-mm-diameter of 12.7 mm) trimmed from HDPE pipes. Obtained from data reported by Krushelnitzky [19].

by a deflection lag factor, e.g., [18]. Similar considerations are required for plastic landfill drainage pipes, although in addition to long time frames, the impact of elevated temperatures on pipe modulus also warrants consideration.

Uniaxial compression tests were conducted on cylindrical specimens cut from a thick-walled HDPE pipe over strain rates between 10^{-3} to $2.5 \times 10^{-6} \text{ s}^{-1}$ to examine the effect of temperature on its stress-strain behaviour [19]. Results from one strain rate and at constant temperatures of 20°C, 50°C, and 80°C are plotted in Fig. 6. The softening of HDPE with increased temperature and reduced strain rate is summarized in Table 1, which reports the secant modulus between 0 % to 5 % strain. The impact of strain rate is most prominent for the specimens tested at 20°C, while at 80°C the softening from increased temperature has a greater affect than strain rate. The data reported by Krushelnitzky [19] was then used to calibrate the viscoplastic model for HDPE developed by Zhang and Moore [15,16] for use at elevated temperatures. The calculated

TABLE 1—Secant modulus for 0 %–5 % strain (MPa) from uniaxial compression tests on cylindrical specimens trimmed from HDPE pipes. Obtained from data reported by Krushelnitzky [19].

Strain Rate (s^{-1})	Temperature ($^{\circ}\text{C}$)		
	20	50	80
1×10^{-3}	360	188	96
1×10^{-5}	232	133	80
2.5×10^{-6}	198	118	75

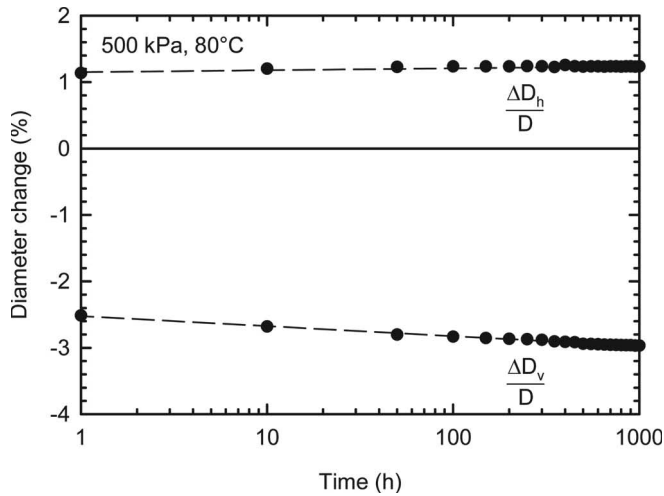


FIG. 7—Measured deflections of a 100-mm-OD, DR-11 HDPE pipe with sand backfill subject to an applied vertical pressure of 500 kPa at 80 °C. Obtained from data reported by Krushelnitzky [19].

stress-strain responses with this calibrated thermo-viscoplastic model are also shown in Fig. 6.

While such tests on the pipe material alone are useful, the actual modulus of the pipe when buried may not be as small as the values in Table 1 because the long-term pipe response will still be controlled by the pipe-soil interactions. Buried pipe tests have been conducted at an applied pressure of 500 kPa for 1000 hrs at temperatures of 22 °C, 50 °C, and 80 °C to provide some longer term data on plastic pipe response when buried [19]. Plane-wall 100-mm-OD, DR-11 HDPE pipes were tested in poorly-graded sand backfill (density index of 30 %). Results from one tests conducted at 80 °C are shown in Fig. 7. The gradual increase in pipe deflection measured with time is attributed to pipe softening. Table 2 gives a summary of results from three temperatures. The rates of increase in defection with time are slower at elevated temperature. This is

TABLE 2—Vertical diameter change (as a percentage of initial pipe diameter) of a 100-mm-OD, DR-11 HDPE pipe with sand backfill subject to an applied vertical pressure of 500 kPa at 80 °C. Obtained from data reported by Krushelnitzky [19].

Time (h)	Temperature (°C)		
	22	50	80
1	–1.69	–2.31	–2.51
10	–2.00	–2.52	–2.68
100	–2.49	–2.71	–2.83
1000	–2.69	–2.82	–2.97

because the pipe modulus starts at a much lower value at 80°C than at 22°C. When the measured results are extrapolated to 50 years, the vertical pipe diameter decrease is estimated to be -3.2% for all three temperatures. Thus when buried, the decrease in pipe modulus from an increase in temperature has a less significant impact on pipe deflections because the soil and its stiffness has a dominant affect on pipe deflections. For pipe design then, appropriate selection of both pipe modulus and soil modulus for temperature and time effects is important. Unfortunately, there is a paucity of long-term creep data on sands. Based upon the longer term buried pipe tests conducted by Krushelnitzky [19], long-term (50-year) pipe modulus of 100 MPa and soil modulus of 30 MPa was inferred for the particular HDPE pipe and conditions tested.

Numerical analysis of these tests using the calibrated thermo-viscoplastic model for HDPE show that although the deflections slowly increase with time, the computed stresses in the pipe decrease with time [19]. Thus for pipe design, provided that short term pipe stresses are kept below material limitations, then pipe stresses will be acceptable in the long-term as they appear to be relaxing.

Large Overburden Pressures

For many small-to-medium size landfills, the depth of waste may range from 20 to 40 m; however, large-to-very-large landfills can be much deeper. For example, Cowland [5] describes some very large landfills with waste depths up to 120 to 140 m. As landfills become larger, there is a need to consider the impact of potentially large overburden pressures on the plastic drainage pipes. The unit weight of municipal solid waste is highly variable—values ranging between 5 kN/m³ to more than 20 kN/m³—have been reported [20]. Consequently, it is not unrealistic to expect vertical overburden pressures approaching 3000 kPa just above the plastic drainage pipe.

Krushelnitzky and Brachman [21] reported a series of experimental results for nominal 100-mm-OD, DR-11, HDPE pipes when buried and subjected to short-term applied pressures up to 3000 kPa. Backfills including a poorly-graded sand (loose and dense), a well-graded gravel (loose and dense), and two loose, poorly-graded gravels—one with smooth subrounded particles; the other with rough angular particles—were examined. Pipe deflections with the angular gravel from four replicate tests are shown in Fig. 8. The pipes deformed in an essentially elliptical manner with a decrease in vertical diameter that was around 40 % larger than the increase in horizontal diameter. Vertical deflections of up to -20% of the pipe diameter were observed at pressures of 2400 kPa. A commonly used deflection limit of 5 % for conventional sewers and culverts [e.g., 12,18] was reached at an applied pressure of only 600 kPa. Despite the large deflections, there was no evidence of buckling or local instability for any of the backfills tested for pressures up to 3000 kPa.

Table 3 provides a summary of measured results for sand and poorly-graded gravel backfills. Of these three backfills, the smallest deflections were obtained for the compacted sand. One might expect the angular gravel to be stiffer than subrounded gravel and hence results in smaller pipe deflections. This would probably be true if both materials were placed at same initial void

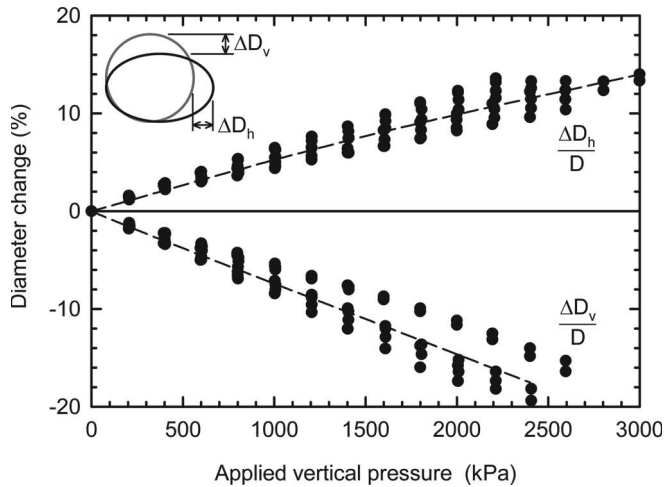


FIG. 8—Deflections of 100-mm-OD, DR-11, HDPE pipe from four replicate tests with angular coarse gravel backfill. Modified from [21].

ratio; but since both materials were placed the same way without compaction, the rough angular particles resisted densification even while being placed; consequently the poorly-graded angular gravel had a lower initial density than the subrounded gravel and hence produced larger pipe deflections. Of the backfills tested, pipe deflections when buried in the poorly-graded angular gravel varied the most because of local variations induced from the relatively large gravel particles.

Finite element analyses of these buried pipe tests were conducted by Krushelnitzky and Brachman [21] to gain insights on the pipe and soil behaviour at very large pressures. They found that the measured deflections could be matched by using geometrically nonlinear analysis with: a viscoplastic pipe

TABLE 3—Short-term deflections of 100-mm-OD, DR-11 HDPE pipe at 22 °C for three backfill materials under large vertical pressure *P*. Inferred soil parameters for use in Hoeg's solution. Data from [21].

Backfill	Pipe Deflections (%)					Inferred Soil Parameters		
		P = 2400 kPa		P = 3000 kPa				
Type	I _D (%)	ΔD _v /D	ΔD _h /D	ΔD _v /D	ΔD _h /D	E _s (MPa)	ν _s	K
Poorly-graded sand	60	−8	4	−10	5	57	0.18	0.2
Poorly-graded subrounded gravel	12	−12	8	−16	10	29	0.1	0.11
Poorly-graded angular gravel	5	−16	12	...	14	20	0.1	0.11

I_D = Density index, P = applied vertical pressure.

model [15,16] to capture effects from pipe yield and time; and a simple Mohr–Coulomb model to account for the influence of shear failure of the backfill, but with a constant Young’s modulus of the sand.

In all tests conducted, the soil modulus did not appreciably stiffen with applied vertical pressures as evident from the fact the rate of increase in pipe deflections did not decrease with increasing applied pressure. Rather an essentially linear increase in vertical deflection with increasing pressure to a certain pressure range, followed by a small increasing rate of deflection with increasing pressure (i.e., reduced stiffness). For example, the measurements in Fig. 8 show that the softening became prominent at applied pressures of 1200 kPa and greater. Although soil stiffness increases with increasing isotropic confining pressure in triaxial compression tests on soil alone [22], when buried along with a pipe, shear strains in the soil from compression of the pipe and biaxial earth pressures may be expected to reduce the soil stiffness. Arguably, the best way to obtain soil modulus values for pipe design under very deep burial is from such buried pipe experiments which include soil-pipe interactions and reproduce soil stresses expected in the field. Inferred secant soil parameters from back analysis of the experimental results up to 3000 kPa are given in Table 3.

Krushelnitzky and Brachman [21] concluded that an HDPE pipe may still be expected to perform its intended function under very deep burial even though the pipe deflections may exceed accepted pipe deflection limits of 5 %, but only provided that the ultimate limit states of buckling and tensile and compressive stress limits are explicitly evaluated. Buckling was not observed in any of the tests conducted. Stresses calculated from the finite element analyses showed either no tension with the poorly-graded sand backfill or tensile stresses below the allowable long-term limits for the poorly-graded gravel. The pipe experienced compressive yield at large pressures and pipe deflections were consequently larger, but this yielding did not cause short-term pipe collapse. At present, there is little data to permit comment on the long-term response of plastic pipes under such large pressures. It may be expected that soil creep may have a more significant impact on long-term pipe deflections for such cases. Consideration of serviceability limits are also required to ensure adequate pipe performance. For example, a plastic drainage pipe in a landfill that has deflected too much such that the passage of pipe inspection and cleaning tools is obstructed would be expected to clog rapidly and hence quickly cease to perform its intended design function.

Service Life

The period of time that the pipe performs its intended design function is its service life. For buried pipes, issues of stress cracking from sustained tensile stresses may not be an issue that could lead to unacceptable performance because they are most probably in a state of coupled long-term creep and relaxing bending stress. Janson [23] postulated that “... constant bending strain the pipe wall is exposed to will not cause failure as long as the pipe material retains its original properties.” For HDPE landfill leachate collection pipes the question

of how long the pipe retains its original properties requires the examination of oxidative degradation in landfill leachate.

Following early stages of the landfill, anaerobic conditions will largely prevail in the leachate collection system [1]; however, reduced oxygen does not mean there is no oxygen and hence it cannot be concluded that oxidation is not an issue that can affect the pipe's service life. On the contrary, the literature on oxidative degradation of HDPE geomembranes in landfill leachates, e.g., [1,24], suggests that oxidative degradation may impact HDPE leachate collection pipe service life. Hsuan and Koerner [25] proposed that the durability of HDPE can be conceptualized by the three stages: (1) the time required to deplete antioxidants, (2) the time required for the induction of polymer degradation, and (3) the time required for the engineering properties of interest to degrade to the extent where it no longer performs its intended design function. Antioxidants are chemical stabilizers that are added to HDPE to limit oxidation during the extrusion and long-term service life of the HDPE. At the very least, knowledge of the time to deplete the protective antioxidants in a HDPE pipes can be used to obtain first order estimates of how long they may perform their function in a landfill.

Experimental results have been reported where antioxidant depletion was measured for one particular HDPE pipe when exposed to air and a synthetic leachate [19,26] that can be used to predict the first stage of oxidative degradation. Specimens of HDPE pipes with an initial crystallinity of 52 %, ASTM E794 [27], were immersed in air and leachate. Although leachates from municipal solid waste can vary widely, the particular leachate consisted mostly of water with a transition metal solution, surfactant (5 mL/L Igepal CA 720), sodium sulfide to act as a reducing agent (to achieve a reduced electrical potential of less than -120 mV) and sodium hydroxide to achieve a pH of 6. This leachate was selected based on a study of municipal solid waste leachate by Rowe et al. [24]. Recent work on antioxidant depletion in HDPE geomembranes has shown that of the various potential constituents in municipal solid waste leachate, it is the surfactant that has the greatest influence on the rate of antioxidant depletion [24]. For experimental control, the leachate was drained and replaced every two weeks. This may simulate period of the landfill with actively flowing leachate in the collection system. This means that the concentrations of the constituents in the leachate were essentially kept constant with time, and thus this may overestimate the rate of antioxidant depletion since the concentration of surfactant may be expected to decrease with time. The experiments were conducted at temperatures of 22°C , 40°C , 70°C , and 85°C . The actual service temperature in a landfill may vary widely depending on the type, composition, and age of waste and landfilling practices [1] and for municipal solid waste landfills may typically be between 10°C to 60°C [1]; it should be noted that for some special industrial wastes, temperatures may approach or even exceed 80°C .

The depletion of antioxidants was quantified by measuring the oxidative induction time (OIT), which is a measure of the polymer's resistance to oxidation. Other factors being equal, the lower the OIT relative to its initial value, the more the antioxidants have been depleted. Both standard and high-pressure OIT tests, ASTM D3895 [28] and D5885 [29], were conducted. Specimens were

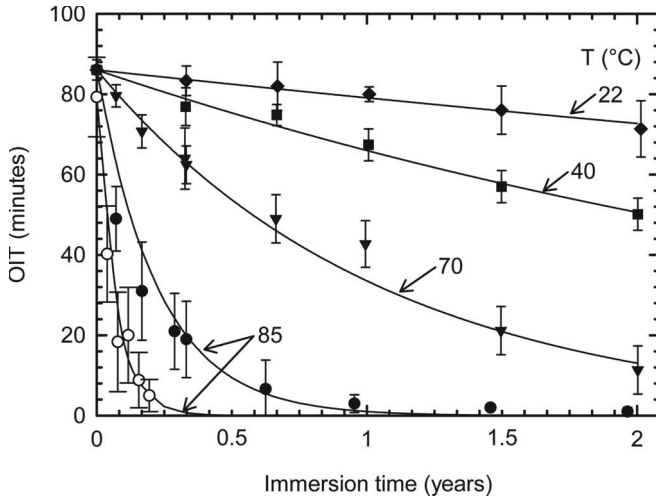


FIG. 9—Oxidative induction time (OIT; ASTM D3895) with time immersed in a synthetic municipal solid waste leachate for one particular HDPE pipe. Solid symbols for 10-mm-thick pipe; hollow symbols for 2.5-mm-thick pipe. Modified from [26].

taken from the inside, middle and outside surface of the pipe and the results were averaged to provide a representative value for the entire wall thickness. Specific details on sampling and testing are reported by Krushelnitzky and Brachman [26]. The decrease in standard OIT with time for pipes immersed in leachate is plotted in Fig. 9. The role of temperature is evident, as there is much faster depletion of antioxidants at higher temperatures. The impact of pipe thickness can also be seen from Fig. 9 as the 2.5 mm-thick pipe showed faster decreases in OIT than the 10-mm-thick pipe tested at 85°C. It is postulated that antioxidant depletion is principally from outward diffusion of antioxidants from the pipe to the surrounding fluid [30]. Consequently, it is logical that depletion is faster at higher temperatures from less resistance to antioxidant diffusion and for thinner pipes as there would be a shorter diffusion path to the pipe surface.

Antioxidant depletion curves from a range of temperatures can be used to infer depletion rates and permit the estimation of the time to deplete the antioxidants using Arrhenius modeling. The results of these calculations by Krushelnitzky and Brachman [26] are shown in Fig. 10 for HDPE pipe specimens immersed in either air or leachate. The impact of chemical exposure conditions is evident as the time to deplete antioxidants is much shorter when the pipe is in leachate than air. This difference is principally attributed to the surfactant in the leachate as the surfactant makes it easier for antioxidants to partition from plastic pipe into surrounding fluid [24]. For the particular thick HDPE pipe tested, it is estimated that depletion of antioxidants may take as long as 600 years at 10°C or as little as 20 years at 50°C when exposed to air, which reduce to 160 years at 10°C or 10 years at 50°C when exposed to synthetic leachate.

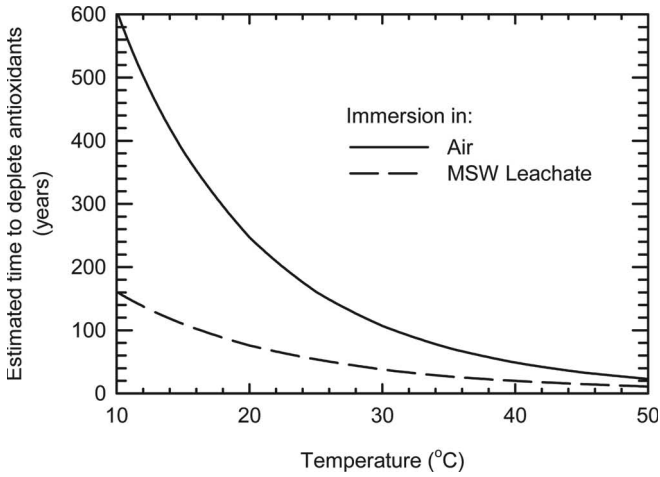


FIG. 10—Estimated time to deplete antioxidants in 10-mm-thick HDPE pipe tested. Modified from [26].

After aging the pipe samples for two years at temperatures up to 85°C, there was no statistically significant change in crystallinity or mechanical properties of the pipe, and hence additional work is required to quantify the second and third stages of oxidative degradation identified by Hsuan and Koerner [25]. For cases where the service life of landfill leachate pipes is required to exceed 50 years, consideration should be given to minimum material specifications of initial standard and high-pressure OIT and retention of a certain percentage of initial OIT after a defined exposure period to ensure an adequate service life.

Summary and Conclusions

Factors influencing the design and performance of plastic drainage pipes in environmental containment facilities were presented. The exposure conditions in a municipal solid waste landfill were shown to be substantially different than conventional sewer and culvert applications, mostly because of chemical and biological clogging in a landfill leachate collection system, but also from potentially large overburden pressures, elevated temperatures and chemical exposure. Hence, these special design issues warrant consideration to ensure adequate performance of plastic landfill drainage pipes.

Perforations for landfill drainage pipes should be as large as possible to reduce the implications of clogging. Circular holes located at the shoulders and haunches minimize the stress concentrations in the pipes. Stress concentration factors obtained from three-dimensional finite element analysis were presented. Coarse gravel backfill surrounding the landfill drainage pipes is required to prolong the time it takes for the voids to clog. The impact of coarse gravel on pipe response can be quantified using design equations that capture the average response of the pipe, which then can be multiplied by empirical factors to

account for local increases in pipe deflection from the variable loading and support provided by the coarse gravel. Values of secant modulus and magnification factors derived from back-analysis of large-scale buried pipe tests were reported for one particular landfill configuration. Landfill pipes may also be exposed to elevated temperatures for long periods of time. Experimental results from and analysis of buried pipe tests at elevated temperatures and longer time frames showed that landfill pipes will have deflections that slowly increase with time, and stresses that relax with time. Experimental and numerical results of landfill pipes under very large pressures were also summarized. It was proposed that deflections larger than usual allowable limits may be accepted, provided that the pipe does not buckle, the long-term stresses are below allowable limits and the deflections are not too excessive such that the serviceability of the pipe is not compromised. For a properly designed pipe that meets both ultimate and serviceability limit states, its service life may be controlled by oxidation. Estimates of the time to deplete the protective antioxidants were presented and showed faster depletion in leachate relative to air, faster depletion with increasing temperature and faster depletion for a thinner pipe relative to a thick one. All of the results presented in this paper are applicable for the particular materials and specific conditions tested; they should not be used directly as a basis for pipe design for conditions that differ from those tested.

Acknowledgments

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Design Development of Large Thermoplastic Chambers for Stormwater Retention

ABSTRACT: Thermoplastic chambers for stormwater retention applications have been in use for many years. One design, an arch-shaped chamber with a 51 in. nominal width and a 30 in. nominal height, has seen widespread acceptance. This paper reports on the development of a structural design for a similar chamber with a 76 in. nominal width and 45 in. nominal height. Development tasks included evaluation of stiffened corrugation designs to permit the use of larger corrugations, evaluation of arch stiffness and stub compression capacity of the chambers, and full-scale field tests of both welded prototype chambers and production chambers. The development program was focused on demonstrating that the chambers met the target safety levels of AASHTO specifications for shallow burial depths with live loads and for deep conditions.

KEYWORDS: thermoplastic, stormwater, chamber, AASHTO

Introduction

The goal of the large-chamber program was to develop a thermoplastic stormwater collection chamber (Fig. 1) to store more water in a smaller footprint

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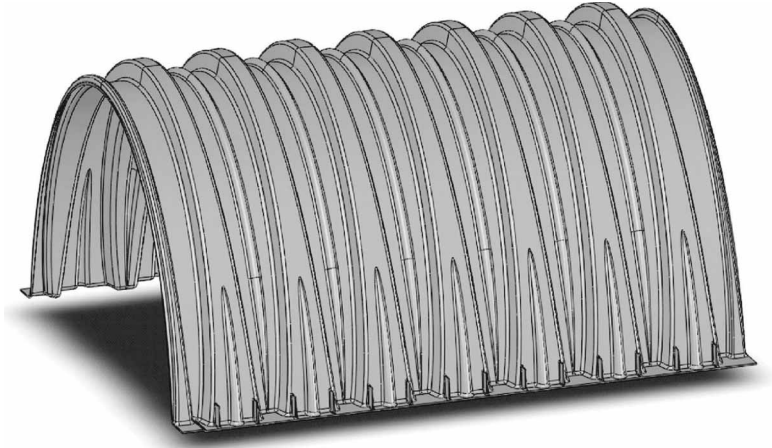


FIG. 1—Large thermoplastic stormwater collection chamber.

than other chambers on the market. As the availability of land in prime building areas has become scarce, consulting engineers are often faced with going deeper to accommodate the mandated stormwater retention volume, often choosing large-diameter pipe or other modular structures that fit more storage in a constrained footprint. The demand for products that fit more stormwater storage in a smaller footprint motivated the initiation of a program to develop a large thermoplastic stormwater collection chamber.

The objectives of the large-chamber development program were (1) to maximize storage volume per footprint area, (2) to meet the structural safety factors specified in the AASHTO LRFD Bridge Design Specifications [1] for live loads and permanent dead loads, (3) to ensure that the product quality and manufacturing consistency necessary to meet the structural objective could be achieved in each part produced, and (4) to be cost competitive measured by total installed cost per cubic foot of storage. The objectives are not independent of each other.

The premise of the AASHTO specifications for thermoplastic pipes is that thermoplastic structures are dependent upon soil support for structural integrity. The stormwater storage volume that can be fit into a constrained space is related to how much soil is necessary to provide the soil strength component of the soil-structure interaction system. Although open graded stone at 40 % porosity is used for the soil component, the quantity of stone required to gain structural capacity diminishes the total storage that can fit in a given footprint, as there is a tradeoff between additional chamber storage volume and the storage volume in the stone between chambers.

The large chamber borne out of this development process has a truncated semi-elliptical arch shape and repeating corrugations, similar to the existing 30 × 51 classification chamber. Installation requirements for the large chamber are also similar to the 30 × 51 classification chamber. This paper presents the structural design and design qualification process for an injection-molded stormwater collection chamber with a 76 in. nominal width and 45 in. nominal height.

Design Development

The large-chamber development program began in November 2004. The first chamber was brought to the market in 2009, completing a 4-year development process. This development process was concurrent with the development of ASTM F2787-09 [2], Standard Practice for the Structural Design of Thermoplastic Corrugated Wall Stormwater Collection Chambers, first published in September 2009. ASTM F2787-09 provides the link between the AASHTO specifications for thermoplastic pipe and chamber design, as discussed in Ref [3].

Development Process

The basic elements of the development process included the following:

- (1) Finite element analysis (FEA) of proposed chamber geometry
- (2) Testing fabricated corrugation and foot shapes
- (3) Testing full-size fabricated prototypes
- (4) Analysis and design improvements
- (5) Mold design
- (6) Laboratory testing of production chambers
- (7) Field testing of production chambers
- (8) Field monitoring of actual installations

Injection molding was selected as the manufacturing process for the large chamber, as (1) injection molding provides consistent and nonvariable wall thickness, which is critical to structural performance, and (2) production rates for injection-molded chambers are higher than other manufacturing processes.

Load Conditions

The target minimum and maximum cover depths were 18 in. and 6.5 ft (78 in.), respectively. The target maximum cover depth was chosen based on the large-chamber fitting in an excavation of the same depth as the existing 30 × 51 classification chamber at its maximum depth of fill. The AASHTO Design Truck is considered for live load in the design of this chamber because the Design Truck creates greater loads at shallow covers and similar live-load pressures at deeper covers as the design tandem. Based on the design cover depths, calculated Design Truck live-load magnitudes at the crown of the chamber are shown in Table 1.

The service-level long-term soil load at the crown of the chamber is about 1.3 psi at 18 in. of cover and 5.4 psi at 78 in. of cover.

At 18 in. of cover, the calculated service-level Design Truck live-load pressure at the crown of the chamber with impact and multiple presence factors is 19.5 psi, and the dead-load pressure is 1.3 psi. The calculated factored live- and dead-load pressures are 34.1 and 2.5 psi, respectively, for a total factored pressure at the chamber crown of 36.6 psi. This is the target-factored pressure for live-load testing, discussed later.

TABLE 1—*Calculated design truck live-load magnitudes at the chamber crown.*

Design Truck Governing Load	Cover Depth (in.)			
	18		78	
	16 000 lb Wheel		32 000 lb Axle ^a	
Live-Load Duration	1 min	1 week	1 min	1 week
Multiple presence factor	1.2	1.0	1.2	1.0
Dynamic-load allowance	1.27	1.0	1.06	1.0
Service live load (lb)	24 400	16 000	40 700	32 000
Live-load distribution area (in. ²)	1250	1250	18 115	18 115
Approximate service live-load pressure at crown of chamber (psi) ^b	19.5	12.8	2.2	1.8

^a The greater cover depth allows for the interaction of multiple wheels on the same axle as the wheel loads are distributed through the soil cover.

^b Pressure values shown here are for reference only and are not actual design values. Live load parallel to the span direction is typically distributed from the ground surface to the chamber through finite element analysis. Distribution perpendicular to the span direction is accomplished following this method when design is by 2D FEA.

Structural Design

Corrugation Profile

Due to the increased loads carried by a larger span, it was necessary to develop a more robust chamber cross-section than the 30 × 51 chamber. The structural capacity of stormwater chambers is evaluated in general accordance with ASTM F2787-09, which adapts the design method for thermoplastic pipes found in AASHTO Section 12.12.3.5.3, “Resistance to Local Buckling of Pipe Wall,” to arch-shaped thermoplastic stormwater chambers. The design method considers local buckling of the corrugation elements of the chamber and is based on the effective-width concept, where the area available to resist compression stress (i.e., the effective width) is reduced based on the element width-to-thickness ratio and the strain in the element as shown in ASTM F2787-09, Eqs 3–5.

This approach has been the basis for design of thin-walled steel sections for many years and was adopted by AASHTO in 2000 for the structural design of thermoplastic pipe. Material strains are compared to the design tension and compression strain limits for the cases of thrust and combined thrust and bending.

Two methods are available to increase the axial compression capacity of a corrugation element (i.e., valley, web, or crest) in the effective-width calculations. The most apparent method is to increase the wall thickness of the chamber, which results in a decreased width-to-thickness ratio and more capacity. This results in increased material volume and product weight. The second method is to increase the stiffness of a corrugation element through the

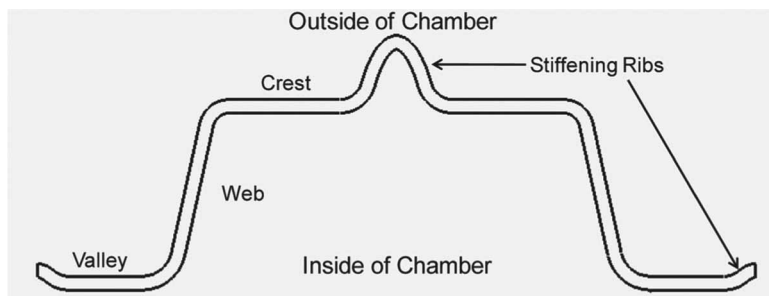


FIG. 2—Cross-section defining corrugation elements.

addition of a longitudinal stiffener, thereby breaking a single corrugation element, such as the crest, into multiple corrugation elements of shorter widths but the same thickness, all with smaller width-to-thickness ratios. This results in a more robust cross-section with minimal added material. A typical period of the corrugation cross-section with longitudinal “stiffening ribs” is shown in Fig. 2.

The interlock of the corrugations of stacked chambers is also a factor in corrugation design to maximize the number of chambers on a pallet and minimize shipping costs. This requires corrugations that taper from the foot to the crown, resulting in a corrugation profile that varies with circumferential location around the chamber.

The variation in corrugation profile around the circumference is shown in Fig. 3, showing two full periods of corrugation. The approximate locations of these cross-sections are shown in Fig. 4. Stiffening ribs are added to the corrugation profile in areas with large width-to-thickness ratios. They are added to the corrugation crest near the foot and taper to a smaller size and terminate at the upper transition area. This increases the capacity of the chamber cross-section in the chamber legs, where the chamber sees the greatest thrust forces. Stiffening ribs also are added to the corrugation valley near the crown and taper down to their termination point near the foot. This stiffens the wider valley elements through the shoulders and near the crown, where the chamber is subject to both axial thrust and bending forces due to live load at lower cover depths.

Equation 5 of ASTM F2787-09 shows the element slenderness factor is dependent on the geometry of the element (width-to-thickness ratio), the strain in the element, and the plate buckling edge support coefficient, k . Typical values of k range from 0.43, the value for a free edge of a long plate under compression loads, to 4 for simply supported edges, and 7 for edges fixed against rotation. Research has determined that a value of 4 is appropriate to define the support derived from two intersecting plates. Where a cross-section element is stiffened by an intermediate stiffening rib, the edge support coefficient is between 0.43 and 4, depending on how effective the stiffening rib is at supporting the larger element. To determine the value for a specific case, the edge support coefficient for the stiffening rib based on its geometry and the geometry of the larger parent element of the stiffening rib in the unstiffened configuration are determined.

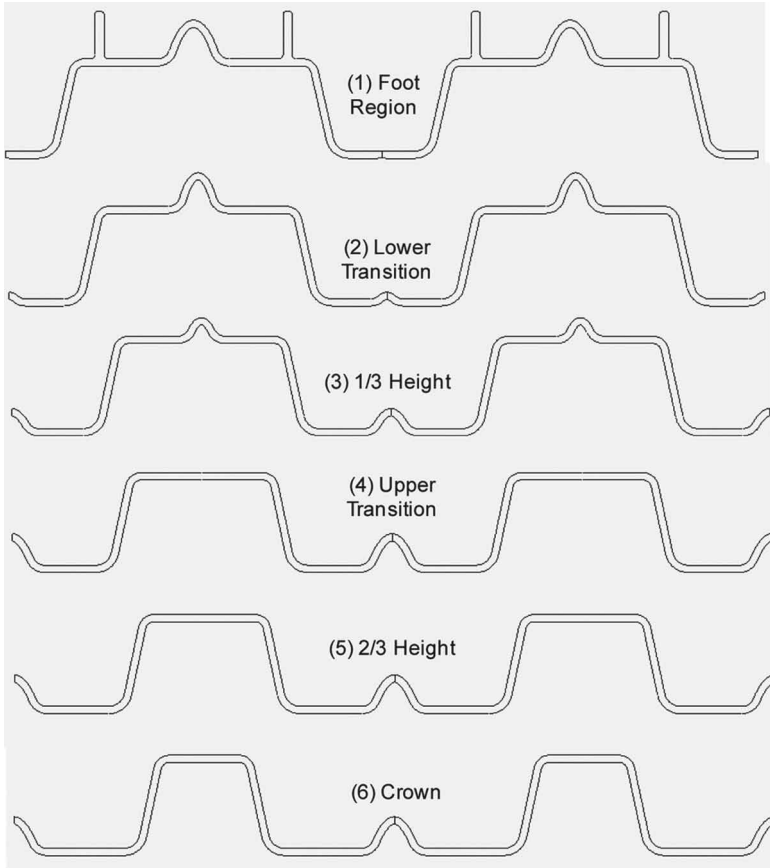


FIG. 3—Chamber corrugation profile at six locations around the circumference.

Details of this calculation are presented in American Iron and Steel Institute's *North American Specification for the Design of Cold-Formed Steel Structural Member* [4]. If the stiffening rib was not sufficient to fully brace its parent elements (for example, the valley elements to either side of the stiffening rib in the crown cross-section), the edge support coefficient of the parent elements was reduced to a value less than 4 but greater than 0.43.

Material Properties

Requirements for the production resin include a minimum short-term tensile strength of 3100 psi and a 50-year tensile strength of 700 psi. Time-dependent material properties used in the design of thermoplastic chambers through material tests on the actual resin used to manufacture the chamber were determined in accordance with ASTM D6992-03 [5]. A limiting compression strain of 3.3 % was established for polypropylene. Based on the tests, we selected three

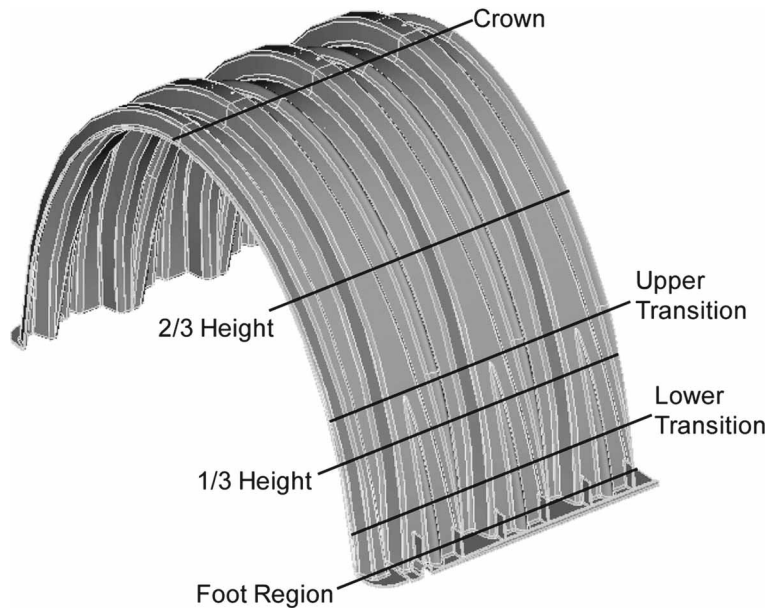


FIG. 4—Chamber corrugation profile locations.

modulus of elasticity values to represent the three live-load durations (discussed below). Values are presented in Table 2.

Finite Element Analysis and Structural Capacity

We modeled and analyzed the large-span chamber with a 0.23 in. wall thickness using the public-domain finite element computer program CANDE [6]. We created models with 18 and 78 in. of soil cover loaded with the AASHTO Design Truck (formerly called HS-20) live loads. The CANDE models incorporated non-linear soil behavior using the Selig/Duncan hyperbolic model, with soil properties developed by Selig for the standard installation direct design concrete pipe installations and later adopted by AASHTO. We modeled coarse-grained foundation and embedment soils with density equal to 95 % of maximum density per the standard proctor test (SW95). The final backfill soils, from the embedment soil to the top of the soil cover, were modeled as granular, well-graded soil

TABLE 2—Creep modulus values for design conditions.

Load Duration	Creep Modulus (psi)
Instantaneous live load (up to 1 min)	158 500
Sustained live load (~ 1 week)	45 000
Long-term (50 years)	27 000

with density equal to 90 % of maximum density per the standard proctor test (SW90). All soils were assumed to have a density of 120 pcf.

We analyzed the chamber for three load durations.

- (1) Instantaneous live load (up to 1 min): Earth load plus AASHTO Design Truck load with impact and multiple presence factors as required by AASHTO [1] and ASTM F2787-09 to account for moving vehicles
- (2) Sustained live load (~ 1 week): Earth load plus AASHTO Design Truck, assumed parked, without multiple presence or impact factors applied, as described in ASTM F2787-09, Section 7.4.4, to account for a parked vehicle
- (3) Long-term (50 years): Earth load only

For the 1-week live load, the live load is not increased for impact or multiple presence since a static vehicle will not induce impact effects and the low probability of having overloaded vehicles parked over the chambers for a long duration. We incorporated thermoplastic creep effects by using an elastic modulus for the chamber representing the stress-strain behavior of the polypropylene under a sustained stress for the duration of that live-load condition.

The model with 18 in. of soil cover (Fig. 5) included three adjacent chambers with 6 in. clear spacing with an applied Design Truck single-wheel load of 16 000 lbs over the crown and separately over the shoulder (at quarter span) of the center chamber. The model had 46 in. of linearly elastic ($E = 3000$ psi) in situ soil below the foundation stone, SW95 foundation and embedment stone from 9 in. below the chamber feet to 12 in. above the top of the chambers, and 6 in. of SW90 backfill.

We increased the depth of the SW90 soil cover for the 78 in. cover model. The live load for this model included the interaction of multiple wheels on the same axle as the wheel loads at the surface distributed through the deeper soil cover. We considered the same two live-load cases as for the shallow case, with impact calculated based on depth of fill and multiple presence effects for a single loaded lane for the 1-min loading and no impact or multiple presence for the 1-week loading.

We used the 50-year creep modulus of 27 ksi to determine the chamber's response to long-term soil loading in the models at both cover heights.

The elemental forces found in the finite element analyses are decomposed into their short- and long-term components, with the short-term component due to the live-load contribution and the long-term component due to the dead-load (soil only) contribution.

The short-term live-load force component for each live-load case is found by removing the soil load contribution from the total results of each short-term (1 min, 1 week) modulus analysis so that only the live-load component of the response remains. The long-term dead-load force component is determined from a separate CANDE analysis at each cover height using the 50-year chamber creep modulus, with soil load only. Examples of the relative magnitudes of the decomposed FEA force outputs at the two cover depths for the Design Truck live load positioned over the crown are shown in Table 3.

The FEA thrust results show that the greatest thrust forces occur due to short-term live loading at the minimum cover of 18 in. However, due to

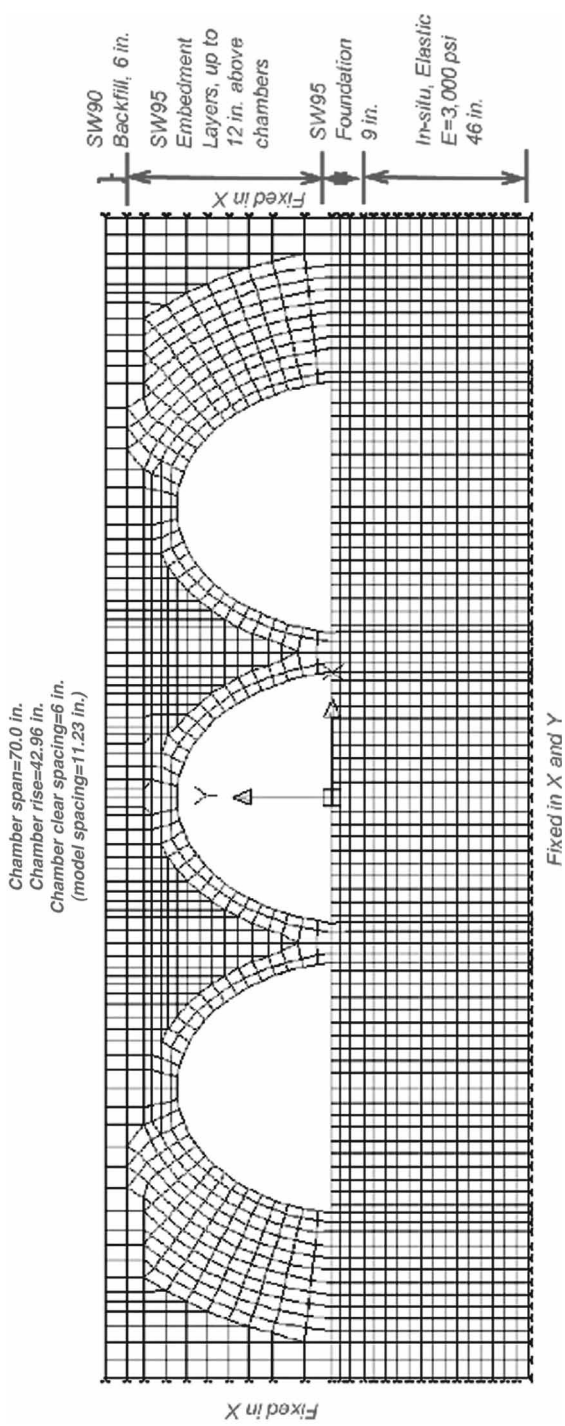


FIG. 5—Soil zones and finite element mesh for the 18 in. cover model.

TABLE 3—Service-level FEA force outputs for design truck wheel over crown.

	Cover Depth (in.)	
	18	78
Dead-load thrust in legs (lb/in.)	36	123
Dead load plus 1-min live-load thrust in legs (lb/in.)	309	165
Dead-load moment at crown (in. lb/in.)	2	5
Dead load plus 1-min live-load moment at crown (in. lb/in.)	620	19

thermoplastic creep effects, design is typically controlled by long-term soil loading plus live load at deeper cover since the sustained load results in a lower elastic modulus for the chamber and higher material strains.

The FEA moment results show a bending moment of 620 in. lb/in. at the crown of the chamber due to short-term live load plus long-term soil load at 18 in. of cover. The bending moment at 78 in. of cover is much smaller since the deeper cover puts the chamber primarily into compression and the live-load effects dissipate at the greater cover depth.

The short- and long-term thrusts and moments are factored and converted to short- and long-term strains acting on the idealized chamber cross-sections. The effective width of each corrugation element is determined as discussed above, and the cross-sectional area is reduced accordingly. The short- and long-term forces are applied to the sections with the reduced effective widths to determine the strains acting on the reduced section. These short- and long-term strains are then summed and compared to the material's limiting compression strain. The ratio of the limiting compressive strain to the applied factored strain is called the structural adequacy of the section. If the structural adequacy is 1.0 or greater, then the required safety factors are satisfied.

The structural adequacies for the two live-load conditions at minimum and maximum cover depths are shown in Table 4.

The controlling structural adequacy for the chamber is 1.05 due to the 1-week live load and long-term soil load at 78 in. of cover.

Tension Strains

Although the overall design is controlled by compressive axial thrust in the deep cover condition, significant bending forces can be developed at the crown of the chamber under live load as discussed above. Maximum bending strains are determined at the extreme outer fiber of the crest and valley from the CANDE

TABLE 4—Structural adequacy at minimum and maximum cover depths.

	Cover Depth (in.)			
	18		78	
Live-load duration	1 min	1 week	1 min	1 week
Structural adequacy	1.27	1.24	1.12	1.05

beam element moment results. Since thrust forces cause compression across the cross-section, the thrust becomes a “beneficial” force in evaluating the tensile forces when summing strains due to thrust and bending at the inside of the valley. ASTM F2787-09 and AASHTO [1] require a minimum dead-load factor for beneficial forces, which AASHTO [1] sets at 0.90, where the beneficial force here is the compressive thrust in the cross-section caused by the long-term soil load. The load factor for live load remains 1.75. The total factored tensile strains at the valley of the crown cross-section due to long-term soil load and crown live load at minimum cover are 1.3 % and 1.6 % for the 1-min and 1-week live-load cases, respectively. With a required 50-year minimum tensile strength of 700 psi and a long-term modulus of 27 000 psi, the minimum long-term tensile strain capacity is 2.6 %.

Bearing Capacity

Traditionally, adequacy in bearing capacity has been determined in terms of safety factors. AASHTO [1] addresses bearing capacity with factored loads and a resistance factor of 0.45, which puts the factor of safety at about 4, with a clause in Article 10.5.5.2.1 that allows for “substantial successful experience ... to justify higher values” for the resistance factor, thereby allowing for lower factors of safety with successful experience. Three bearing capacity design conditions for chambers are discussed in Ref [3] and are addressed below in terms of factors of safety.

Bearing of Chamber Foot

Axial thrust in the chamber legs is transferred to the foundation stone through the chamber foot. The chamber foot must transfer load from the chamber leg into the foundation stone. Factors of safety for foot bearing on the foundation stone are about 3.0 at moderate cover depths. At minimum cover, where live load contributes significantly to the overall load at the foot, the lowest factor of safety is 2.6. Although lower than typical factors of safety for bearing capacity, the foot capacity is deemed acceptable, as moderate settling of the foot into the foundation stone will reduce the load in the chamber through arching, and as noted in the following section, the stone column between the chambers is evaluated for the total load, including that carried by the chamber leg. Thus, there is little risk to the overall installation. Also, the factor of safety for this condition is similar to that in the 30 × 51 classification chamber, which has performed well for several years.

Stone Column between Chamber Feet

Total load from the soil between the crowns of adjacent chambers up to the ground surface, including the soil column between the two chambers and vehicular live load, acts on the foundation stone surface between the feet of the two adjacent chambers. For this condition, the factor of safety is over 5 at minimum cover and decreases to 3.2 at 78 in. of cover.

Demand on Native Soil below Foundation Stone

The demand on the native soil below the foundation stone can be found by taking the applied pressure at the foot level between the chambers and distributing it through the foundation stone to determine a uniform pressure demand on the native soil subgrade. This is a function of native soil conditions and is thus a responsibility of the site engineer. Low native soil bearing capacity can normally be addressed through the use of a thicker foundation stone layer.

Global Buckling

Global buckling of these chambers is evaluated generally following the method in ASTM F2787-09, Section 8.5.2. This design equation is under consideration by AASHTO for the design of thermoplastic pipe.

The chamber's global buckling adequacy is determined as a ratio of critical buckling thrust to the maximum factored applied thrust. The controlling global buckling adequacy is a function of both the applied thrust and the moment of inertia of the cross-section where the thrust occurs, considering all load cases and durations. Global buckling is evaluated conservatively using the 50-year chamber creep modulus.

Using a 50-year creep modulus of 27 000 psi, the minimum global buckling adequacy for the chamber is 1.34.

Serviceability

There are two serviceability (vertical deflection) considerations for stormwater retention chambers. The first is limiting vertical deflections to minimize ground surface deformations, and the second is to maintain the design hydraulic storage capacity of the chamber. ASTM F2787-09 limits vertical deflections to 2.5 % of the chamber rise, or 1.03 in. for this chamber.

At 18 in. of cover, the service-level dead-load vertical deflection is 0.13 in. and the service-level live-load vertical deflections are 0.90 and 0.93 in. for the 1-min and 1-week live loads, respectively. The maximum service-level vertical deflection at 18 in. of cover due to dead and live loads combined is 1.06 in.

At 78 in. of cover, the service-level dead-load vertical deflection is 0.45 in., and the 1-min and 1-week crown loads result in service-level live-load vertical deflections of 0.06 and 0.10 in., respectively. This results in a maximum service-level vertical deflection of 0.55 in. at 78 in. of cover. The total service-level vertical deflection at 18 in. of cover is greater than the allowable limit specified above. Vertical deflections are also verified during live-load testing as described below.

Design Qualification Process

Production Wall Thickness

Ongoing wall thickness measurements from the initial production to current production demonstrate that the design wall thickness of 0.230 in. is achieved within acceptable tolerances. Chambers measured during the spring of 2009

demonstrated an average thickness for a chamber ranging from 0.229 in. with a standard deviation of 0.007 and coefficient of variation of 3.0 % to an average of 0.230 in. with a standard deviation of 0.007 and coefficient of variation of 2.9 %. A minimum of 210 measurements was taken on each of three sample chambers.

Stub Compression Tests

We conducted stub compression tests to determine the compression capacity of the cross-sections in general accordance with the test method in Appendix G of National Cooperative Highway Research Program Report 631 [7]. We took specimens from the foot region, lower transition, and crown sections of the chamber.

The strain capacity of the section (strain at maximum load) must be greater than the 3.3 % limit used in the structural adequacy calculations described above. Figure 6 shows load versus strain curves for the stub compression testing. All specimens met or exceeded the 3.3 % strain limit.

Arch Stiffness Tests

We performed arch stiffness tests in accordance with ASTM F2418-09 [8], Section 6.2.8. The seven specimens were approximately three periods in longitudinal length, cut just short of the valley stiffening rib.

The average arch stiffness constant at 2 % deflection was 273 lb/ft/% deflection, with a minimum test value of 267 lb/ft/% deflection. The average deflection at peak load was 8.4 %, with a minimum test value of 8.2 % deflection at peak load. Target minimums for the arch stiffness constant and maximum deflection were 250 lb/ft/% deflection and 7.0 % deflection, respectively.

Live-Load Test

Twelve large chambers were installed with soil and material zones similar to standard installation requirements. The test installation differed from standard requirements in the following ways: (1) Minimum cover was 18 in. or less to increase live-load effects and to demonstrate a factor of safety (see Ref [3]) and (2) adjacent rows of chambers were conservatively placed with no clear spacing between chambers so that chamber feet from adjacent rows were in contact. Since a final production end cap was not available at the time of testing, a thermoformed end cap was used in these tests.

The chamber test bed was laid out to include three rows of chambers so that the center row of chambers would represent an actual installation as closely as possible. Also, three chambers were installed perpendicular to the three rows so that the live-load vehicle would traverse some chambers across the span and others parallel to the span. Chambers were monitored for deflection and were visually inspected.

The first 6 in. of cover over the tops of the chambers was crushed stone or concrete granular embedment material. Two 6 in. lifts of well-graded granular soil were added over the crushed stone and compacted with a vibratory plate compactor to bring the total cover over the top of the chambers to 18 in.

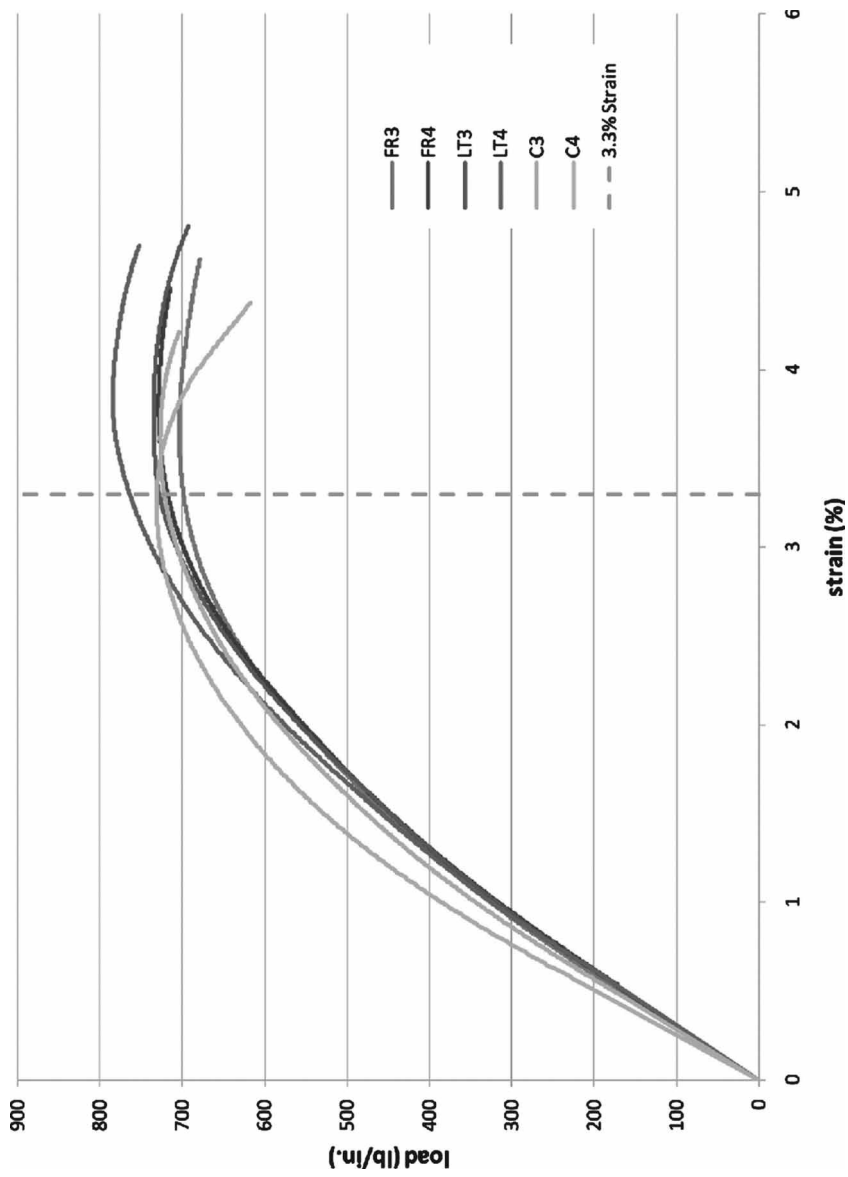


FIG. 6—Stub compression load versus strain curves.



FIG. 7—Live-load testing of chambers and end caps.

Live-Load Test Procedure

Live loads included static and dynamic vehicle axle loads ranging from 27 000 to 35 000 lbs (Fig. 7). Static and dynamic live-load tests were conducted. For a static test, we positioned a wheel over a chamber or end cap for 10 min. We conducted static tests over the crown and shoulder of instrumented chambers and above an instrumented end cap. For dynamic load tests, the wheel made three passes over the crown and shoulder of the instrumented chambers and an instrumented end cap. We also subjected the instrumented end cap to a forward, stop, and reverse moving load at an angle.

We applied this test procedure with the 35 000 lb axle at 18 in. of cover, then reduced the cover to 12 in. and repeated the procedure, first with the 27 000 lb axle, then with the 35 000 lb axle.

As a final test, the crown of the chambers and end cap were subjected to 100 passes of the 35 000 lb axle load at low speed at 12 in. of cover.

Significant rutting occurred during the testing, particularly during the 100-pass test, where the soil cover decreased significantly between the chambers and to as little as 4 in. over the end caps. Cover over the crowns of the chambers remained at about 11 in., causing a wavelike profile at the ground surface as the wheel moved over adjacent rows of chambers.

Chamber Performance during Live-Load Tests

The peak downward movement of the crown of the two instrumented chambers during a single-load event was typically 1 in. or less. Throughout all load events,

on two occasions one of the instrumented chambers registered deflections greater than 1 in., with the greatest deflection of 1.3 in. occurring during the first load cycle as the chamber and stone were worked into place. Vertical deflections mainly occurred as the wheel passed over the chambers, with the chambers rebounding after the wheel moved off the chambers. This verified the vertical deflections calculated in the CANDE analyses and ensured that deflections are generally 2.5 % of the chamber rise or less.

There was cumulative downward movement throughout the loading cycles up to about 1 in. This movement was most likely due to the chamber and embedment material being worked into place throughout the loading events. Vertical movement is measured as absolute crown movement, relative to the fixed base of the draw wires; this vertical movement includes contributions from the chamber bending at the crown, as well as possible settlement of the chamber feet into the foundation stone throughout the load cycles. This settlement resulted in part from the increased live load due to the rutting noted above. This rutting is not expected in a permanent paved installation.

Approximate measurements of the penetration of the chamber feet into the foundation stone were taken by placing a straight edge on the instrumentation table across the width of the chamber and measuring the change in elevation of where straight edge intersected the chamber legs. After completing the load cycles at 18 in., the foot penetration was 0.8 in. After all load cycles, including the 100 passes at low cover, the maximum foot penetration was about 2.9 in., occurring on the foot that was subject to repetitive loads over the shoulder of the chamber.

End-Cap Performance during Live-Load Tests

Under loading at the initial depths of fill (18 and 12 in. over top of chambers), the thermoformed end cap typically showed less than 1 in. of inward displacement for a single-loading event. The end cap exhibited cumulative inward movement, similar to the chambers, during the repetitive loading.

Under the most severe condition, 4 in. of cover over the end cap loaded by the 35 000 lb axle, the center portion of the end cap buckled inward. This buckling occurred at loads well in excess of the factored load and did not result in a breach of the end cap or damage to the chamber. However, due to the end-cap buckling, the minimum cover for the chamber installations has been increased from 18 to 24 in. until a final production end cap is manufactured.

Deep Burial Test

Nine prototype large chambers were installed in a three by three grid under 19.5 ft of fill in December 2007 (Fig. 8). The prototype chambers were handmade from welded pieces cut from extruded sheets of polyethylene. Three of the chambers had instrumentation including strain gages and draw wires, and four soil-pressure cells were installed in the embedment stone. Data is monitored remotely, and field visits are performed periodically to observe chamber performance through man-access pipes. Backfill was placed in ~ 12 in. lifts and

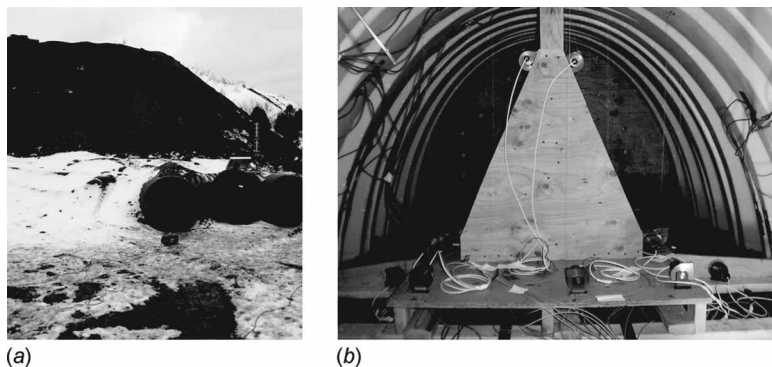


FIG. 8—(a) Access pipes and soil cover over chamber installation, (b) instrumented center chamber under 19.5 ft of fill.

compacted, with backfill density monitored through nuclear density gage measurements. The average wet density of the backfill was 124 pcf.

After nearly 2 years of data monitoring, the three instrumented chambers show evidence of the anticipated thermoplastic creep effects through continued deflection and straining. Strain magnitudes are of the order of 0.5 % at the crown and 1–2 % in the legs near the feet. Vertical deflections are of the order of 4 %.

Conclusion

As land in prime building areas becomes scarce and environmental regulations more demanding, it is necessary to look toward larger stormwater retention structures to fit more retained volume in a smaller footprint. The wide acceptance of the existing 51 in. nominal width arch-shaped chamber design suggested a larger span with the same characteristics could help fill this need.

The design requirements of ASTM F2787-09, which links the AASHTO structural design specifications for thermoplastic pipe to open-bottomed chambers, provides a sound basis for design of larger chambers. Important design considerations include the following.

- Understanding live and dead-load magnitudes over the range of cover depths
- Development of a more robust chamber cross-section to resist the greater loads in the local buckling failure mode—including consideration of intermediate stiffeners
- Consideration of thermoplastic creep effects to properly design for the given loads
- Methods to accurately determine design forces through finite element modeling with soil-structure interaction
- Design qualification to provide verification of the computer models through final product measurements, laboratory tests, and full-scale field installation tests.

Execution of these important considerations has demonstrated that the newer 77 in. nominal width stormwater retention chamber meets the target safety levels set by AASHTO for long-term soil loading and short-term live loading for depths of fill between 18 and 78 in. when installed in accordance with the manufacturer's recommendations.

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Technical Considerations When Fabricating PVC Pressure Fittings*

ABSTRACT: Unlike the fully automated production of polyvinyl chloride (PVC) pressure pipes and injection molded fittings that involve large production volumes, manufacture of fabricated PVC pressure fittings is labor-intensive and involves short production runs. Fabricating PVC fittings is as much an art as it is a science. It is important to understand the steps involved in the manufacture process to appreciate the considerable amount of engineering and design that goes into their production. Quality control measures that are in place, as well as the importance of meeting various applicable industry standards, are discussed. Development of various unique configurations of fittings as well as advances in technology within the industry, such as an integral joint restraint mechanism, is also discussed.

KEYWORDS: PVC fabricated fittings, fitting manufacture, tees, wyes, bends, butt fusion, fiberglass, integrated joint restraint

Introduction

Manufacturing polyvinyl chloride (PVC) fabricated fittings is quite different than the manufacture of PVC pipe. With pipe, the production runs are large, and there is a great deal of automation. With fabricated fittings, the production runs are short, and the manufacturing process is labor intensive. To understand

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the quality built into a fabricated PVC fitting, one must first learn the various manufacturing steps involved. This paper introduces those manufacturing steps, as well as the quality control (QC) measures and engineering considerations for each of those steps.

Applications

In addition to being labor intensive, the manufacturing process for fabricated fittings is very flexible. This allows a single manufacturing facility to produce fittings for a broad range of applications. Some of the applications in which fabricated PVC pressure fittings are used are buried water systems, buried storm sewer systems, buried and above ground irrigation systems, process piping and air ducting in industrial applications, plumbing applications, dual-containment systems, and vacuum and force-main sewer systems.

Configurations

The flexibility of a fabricating fitting operation is revealed by the wide range of configurations available: tees, wyes, crosses, bends, sweeps, couplers, reducers, tapers, caps, plugs, and saddles. Besides these standard types, there are also custom fittings, with few limitations to their configurations.

Illustrative Examples

This paper limits itself to fabricated pressure fittings. This application best lends itself to illustrating the various production steps that may be needed to produce a fitting. Additionally, three different example fittings have been selected to demonstrate a variety of production processes and include a 5° bend, a 45° bend incorporating joint restraint mechanisms, and a tee.

Fabricated Fittings Production Departments

The production departments at a large PVC fitting fabricator are listed in the following and briefly described.

- *Raw materials:* The raw material for producing fabricated PVC fittings consists mostly of PVC pipe. The other raw materials needed for the production of the three fittings illustrated in this paper are gaskets, Bull-DogTM grip rings and casings (BullDog is the brand name for a product line that has restraint hardware built into the bell of the product), primer, solvent cement, fiberglass mats and roving, polyester bonding resin, and ductile iron flange rings.
- *Cutting area:* In most cases, the PVC pipe is cut to a particular length in order to produce a fitting. This is the cutting department's function. Depending on the fitting, the ends may be cut square, to an angle, or to a compound angle. The cutting department also routes holes in the pipe, which later accommodates the branch of a tee or wye. Beveling spigot ends to the proper depth and angle is performed here as well.

- *Heat forming*: PVC is a thermoplastic. That means it can be reheated and reshaped multiple times. The other type of plastic is a thermoset. Once formed, a thermoset cannot be reshaped. The heat forming department is quite inventive at using PVC's thermoplastic properties to reshape a straight section of pipe into a thermoformed bend or to form a pulled hole which will accept the branch section of a tee or wye. Belling or socketing the ends of a fitting, and installing the Bull-Dog internal restraint, are also performed by this department. The heat forming department is equipped with glycerin tanks, glycol tanks, silicon heating blankets, dry/radiant heat ovens, and numerous presses and bellers.
- *Assembly*: This department is responsible for connecting the branch outlet into the body of the tee or wye or cross. The assembly department is also involved when two smaller angled bends are connected to make one larger degree bend. Connections are made through solvent cement, hot air welding, or extrusion welding. Butt fusion is one connection method that falls outside the purview of the assembly department. Instead, butt fusion is a department of its own.
- *Butt fusion*: This is a quick and effective method for connecting two sections of pipe together as long as the fusion parameters are correctly adjusted for the thermoplastic being joined. An entire department is dedicated to this operation in order to accommodate typical fitting production volumes.
- *Fiberglass*: Fabricated pressure tees and crosses, as well as wyes and other configurations, often need to be reinforced in order to achieve the desired pressure rating. The amount of external reinforcement depends on the pressure capability desired, the diameter of the fitting, and the fiberglass raw materials used. Fiberglass reinforcement consists of mats, roving, and polyester bonding resin.
- *Machining*: This department is responsible for items like flanges, which need to have holes drilled, the surfaces faced, and shoulders counter-sunk. Threading as well as custom components and configurations are also machined in this department.
- *Quality control*: Given the variety of processes involved in making fittings and the level of worker involvement in the finished product, the QC department has a great many tasks to perform throughout the production process.
- *Inventory, packaging, shipping*: These operations are self explanatory and familiar to most. As such, it will only be given cursory attention in this paper.

Production Steps for Three Types of Fittings

It is necessary to have all the above-listed departments to produce a range of fittings in different diameters, configurations, and for various applications. A particular fitting; however, rarely visits every department. As a group, though, the three examples will involve every department mentioned earlier.

Five-Degree Bend

The first example was selected for its simplicity, and involves a spigot by bell, 8 in. diameter bend, made from DR18 C900 pipe. By following the production of the 5° bend from its raw materials to the finished product, one will gain greater familiarity with some of the production departments within a fabricated fitting facility.

- *Raw materials/quality control:* The raw material for producing this bend consists of AWWA C900 DR18 PVC pipe and one Rieber gasket. A Rieber gasket is a metal-reinforced rubber seal used in PVC pipes. The QC department checks the applicable dimensions of the pipe, as well as checking its appearance. Voids, cracks, or inclusions would be examples of defects that would justify rejection of the PVC stock pipe. For the gasket, the QC checks consist of verifying the compound. An SBR compound is typically used unless the purchaser has requested a specialty gasket. The appearance of the gasket is checked as well. Blisters, cracks, or excessive bloom would be cause for rejection.
- *Cutting area/quality control:* The next stop for the in-progress bend is the cutting area. One end is cut to square and the other end to a 5° angle. QC checks involve verifying the angles of the cuts and the cut length. The end that will become the spigot end is beveled by a router.
- *Heat forming:* The 5° bend is a thermoformed bend. The bend is made while the female end is belled. To prepare the female end for belling, that end is placed into a glycol tank and heated to between 270 and 300°F. It is then fixed to the belling press. The warmed end is down, and the top plate of the unit is 5° from vertical. When the pushing plate descends vertically, it pushes the material over the tooling, forming the bell and, due to the angle of the top pushing plate, simultaneously bends the warmed section to the proper angle of 5°. Clamps hold the material tight on the tooling mandrel from the exterior while vacuum chambers in the mandrel pull the hot PVC around the gasket, locking it in. The fitting is then cooled with water or air to maintain its new shape. The finished product is shown in Fig. 1.
- *Quality control:* To confirm the overall production process, and consistent success with it, finished product samples are subjected to pressure tests. The sample frequency is driven by industry standards, customer specifications, and internal policies. An example of one such test would be a 2-h test at twice the fitting's pressure rating. Besides QC testing, designs are proven through qualification testing.
- *Inventory, packaging, shipping:* This is the fitting's final stop.

Forty-Five-Degree Elbow

The second example is also a bend, but it is a bit more complex. It is manufactured using a 12-in. diameter, DR14 C900 stock pipe. A different production process is used to construct the bend, and the bell ends will have restraint hardware installed during the belling operation.



FIG. 1—Five-degree, male-by-female, bend.

- *Raw materials/quality control:* Most of this discussion is similar to that for the 5° bend previously discussed. In addition to checking the pipe and gaskets; however, the restraint mechanism, Bulldog grip rings and casings, will need to be checked. Bulldog is the brand name for a product line that has restraint hardware built into the bell of the product. QC checks for the Bulldog hardware include inspecting the hardware to ensure that it has been e-coated and Teflon coated as required, that there are no blisters in the e-coat which would be an indicator of excessive porosity of the underlying ductile iron, and that proper size of grip ring and casing were provided.
- *Cutting area/quality control:* As with the 5° bend, the next stop for the 45° elbow is the cutting area. This time, the cuts are both square. QC checks involve verifying the angles of the cuts and the cut length.
- *Heat forming:* One end of the fitting is prepared for belling by placing it into a glycol tank and heating it to between 270 and 300°F. Then it is fixed to a belling press so that the end being belled is vertical. This time, instead of only loading a gasket onto the belling mandrel, a gasket and a Bulldog casing is loaded. The beller then forms the bell around the gasket and casing with vacuum. The process is repeated to make the other internally restrained bell.
- *Cutting area/quality control:* The section of pipe is now equipped with a bell on each end. Each bell has a gasket and a Bulldog casing. The double-bell barrel is now split across its midpoint at 22.5°. QC checks involve verifying the angle of the cuts and the cut length.
- *Butt fusion/quality control:* Next, two pipe segments are joined together by the butt fusion department. The two 22.5° cuts are set square and opposite one another. The two segments are clamped onto the butt fusion machine. The ends are faced, heat soaked, and fused. It is important to

note that the fusion parameters for PVC are different than those for HDPE. Figure 2 shows the resulting 45° BullDog by BullDog fitting.

- *Quality control:* The grip ring portion of the BullDog restraint is installed into the casing here, and inspected. A routine QC check is a pressure test of the finished product which validates the production process as a whole.
- *Inventory, packaging, shipping:* The bend then waits to be shipped to the customer.

Sixteen-Inch Tee

The third example is the most complex of the three. The tee will be made from SCH80 ASTM D1785 PVC pipe stock and have a 16-in. diameter running leg and a 14-in. branch. Tees, in general, are more a demanding configuration because of the hole made in the body of the tee for the outlet. Only crosses and wyes are more difficult. For pressure fittings, the hole in the tee's body makes achieving the desired pressure rating more challenging. This is solved with fiberglass reinforcement. Adding to the complexity of this fitting is the flanged connection specified for the outlet, which requires the fabrication of a separate component for final assembly.

- *Raw materials/quality control:* Most of this discussion is identical to that for the prior examples. In addition to pipe for the run of the fitting, this tee will need additional sections of pipe to form the flanged branch. It will also need the internal components for pressure tees of this size: a flange backing ring, solvent cement, primer and fiberglass mat, roving and chemicals. Monitoring the solvent cement is important. If it has become lumpy, it has spoiled and must be replaced. Inspecting solvent cement is discussed in standard specifications for the product.

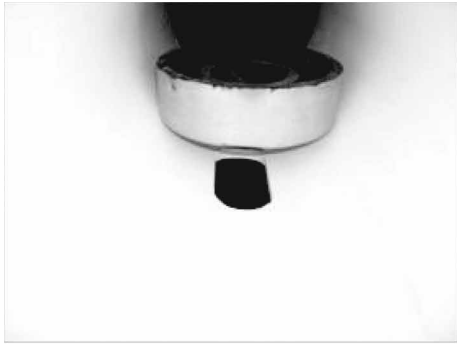


FIG. 2—Forty-five-degree elbow, female-by-female, with BullDog restraint mechanism.

- *Cutting area/quality control:* As before, the first stop is the cutting area. The running leg has square cuts on both ends. Before leaving the cutting area; however, a hole is routed into the running leg with the aid of a metal template, Fig. 3. The routed hole looks strange after leaving the cutting area. The logic of the routed hole will become clearer after a swedge has been pulled through the hole in the heat forming department. The other major component, the 14-in. outlet, requires three pieces of 14-in. SCH80. Both are cut square and to length.
- *Heat forming:* Prior to forming the pulled hole for the 14-in. outlet, the area around the routed hole is heated with a silicon heating blanket. The running leg is affixed to the press, and the pulling swedge is connected to the press while it is inside the 16-in. SCH80 section. The press pulls the swedge through the routed hole to form a pulled hole, and then the pulled hole is cooled, Fig. 4(a), 4(b). Before leaving the department, both ends of the running leg are socketed.
- *Assembly:* The stiffener is solvent welded into the pulled hole, Fig. 5. *Note:* The process for making the solvent welded seam for this product has been pre-qualified and has already demonstrated that it provides a lap shear strength that exceeds 900 psi.
- *Heat forming:* Then the 14-in. branch is pressed over the stiffener until it butts up against the surface of the 16-in. run, forming a saddle of sorts. (In the industry, it is known as a spout.) The end of the 14-in. branch was heated in glycol prior to pressing it over the stiffener, Fig. 6. Separately, one section of 14-in. pipe is heated and formed over tooling, equipping one end with a flanged face.
- *Machining/quality control:* At the machine department, the PVC section equipped with a flange is faced with a groove pattern allowing for a

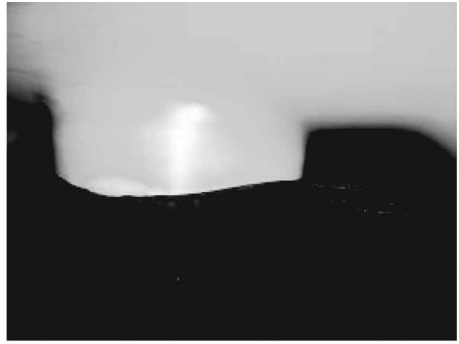


FIG. 3—Hole routed into 16-in. running leg. The hole will later be thermoformed into a pulled hole.



Running leg with routed hole and swedge tool used to form the pulled hole

(a)



Swedge tool forming the pulled hole

(b)

FIG. 4—(a), (b) *Thermoforming the pulled hole.*

superior flange face gasket seal. QC checks to ensure that all dimensions are within tolerance.

- *Assembly/quality control:* Everything comes together for the tee at the assembly department. The branch outlet is solvent welded to the stiffener and running-branch. Next, the ductile iron (DI) flange ring is properly aligned and the 14-in. flanged section of PVC is solvent welded to the branch while passing through the DI ring.

Note: As with solvent welding the stiffener into the pulled hole, the solvent welded seams made in this production step have been pre-qualified and have already demonstrated that they provide a lap shear strength that exceeds 900 psi.



FIG. 5—*Pulled hole with stiffener.*



FIG. 6—A spout has been thermoformed over the stiffener.

- *Fiberglass:* In order to achieve a rating of 220 psi required by the customer, the tee will need to be reinforced with fiberglass. The fiberglass is applied via a hand lay-up. First, a special bonding resin is applied. That is followed by two layers of mat and roving. A mat is a coarse fabric sheet that is made from randomly placed strands. Figure 7 shows a photo of the mat material.

The roving is the material that gives the reinforcement its strength. It is a collection of strands of glass wound into a cylinder without much twisting of the strands. Additional layers are added as needed for the size of the fitting and the pressure rating desired. Roving layers and mat layers are staggered and always have a resin binding layer between them. The sequence is (1) mat-resin-roving-resin, (2) mat-resin-roving-resin, etc. Each layer is hand laid on the fitting and rolled out with a roller to form a perfect shell for the PVC fitting. It is important to remove any air that gets trapped under the layers. For aesthetic reasons, the final step in this department is to sand off any strands or sharp



FIG. 7—Mat material.

edges on the ends of the fitting. Then apply gel-coat for the appropriate color. Blue is the color typically selected for potable water applications. After leaving the fiberglass department, the tee is now considered a thermoset material rather than a thermoplastic because of the fiberglass reinforcement.

- *Quality control:* As with the other examples, a statistical sample is pulled and pressure tested to verify overall quality and integrity.
- *Inventory, packaging, shipping:* Every fitting's last area.

Selecting Production Methods

The three examples discussed how the fitting was made, but it did not cover why that particular fabrication method was selected over the other options available. In some instances, the production method selected is customer driven, and in other cases it is application driven. Or, the method may have been driven by equipment capabilities or for engineering reasons. The complexity of the operation precludes a comprehensive discussion, but two production issues will be reviewed in order to cover this at a cursory level.

Joining Options

A fitting fabricator has many options for making bends.

- The bend may be thermoformed, Fig. 8(a).
- Two thermoformed bends may be solvent welded together to make a bend with a larger angle, Fig. 8(b).
- Two or more mitered sections may be butt fused together to make the bend, Fig. 8(c).
- Two or more mitered sections may be hot-air-welded together or extrusion welded together to make the bend, Fig. 8(d).

Thermoforming is the most preferred option. This method produces a bend out of a single piece of material, which eliminates the necessity of a seam between two sections of PVC. Thermoforming is used whenever possible, which is up to 45°. When using this production method, the fabricator must be conscience of the wall thinning occurring on the outside of the bend. For larger angle bends, the fitting may need to be made out of PVC that is thicker than the pipe it is joined to. By using thicker walled PVC, the fitting will be able to meet the pressure rating of the pipe.

After thermoforming, solvent cement is most preferred because of the quality and strength of this type of chemical bond. To form the bend, two thermoformed bends are manufactured. The thermoformed bends have a bend angle half that of the finished elbow. To connect the two thermoformed bends together, each is socketed. The two sockets are then solvent welded to the same stub of pipe. One example might be the joining of two 45° bends to achieve a 90° line deflection.

Butt fusing is driven by end user as well as engineering reasons. Bends fabricated with butt fused seams will most often be seen in industrial applications as that customer favors that method of fabrication.



8a, Thermoforming



8b, Solvent-welding



8c, Butt fusion.



8d, Extrusion welding. (This fitting has been prepared for fiberglassing in the area near the extrusion weld.)

FIG. 8—Manufacturing options for making a bend.

Extrusion welding and hot-air-stick welding are other methods for welding mitered sections of PVC together in order to make a bend. These joining methods tend to be used more on non-pressure applications, and with bends intended for use with profile-wall non-pressure pipe. In some cases, the application of fiberglass reinforcement can achieve required pressure ratings.

Heating Options

Glycol and glycerin tanks are not the only means available for heating PVC to temperatures where it becomes pliable. Indeed, PVC pipe plants use radiant heat to prepare the pipe's end for the belling operation. Why do fitting fabricators use tanks? Liquids are used for two reasons. The glycol and glycerin heat the PVC more quickly, more uniformly, and result in better heat retention and a longer window of time for performing the thermoforming operation. The second reason is the greater flexibility for heating only selected areas of the fitting. Only the part of the fitting that is immersed will be heated to thermoforming temperatures. Fitting fabricators do use radiant heat, when necessary, for particular operations.

Conclusion

The fabricated PVC fitting industry is now over 30 years old. Over the past three decades, this industry has refined its manufacturing techniques, improved its QC procedures, and broadened the range of applications in which its products are used. With this history, experience, and expertise, this manufacturing technology can no longer be considered new or untried. It is now a mature and proven technology. To increase awareness of the quality and engineering built into the product, this paper introduces typical manufacturing steps in the production of fabricated fittings, touched on some considerations when deciding which manufacturing process to use, and emphasized the quality checks made at every step of the fitting fabrication process.

Amster Howard¹

Deflection Lag, Load Lag, and Time Lag of Buried Flexible Pipe*

ABSTRACT: The Iowa formula was published by Professor M. G. Spangler of Iowa State Univ. in 1941 to predict the deflection of buried flexible pipe. In the equation, he used the term “deflection lag” to describe the increase in flexible pipe deflection after the maximum load was reached over the pipe. Unfortunately, the term is currently improperly used in ASTM, AWWA, and ASCE manuals and standards to reflect the increase in deflection starting the day the backfill is completed over the pipe, rather than when the maximum load occurs. The maximum load on a pipe is generally not attained until after 3–12 months after construction. The history of the deflection lag term and the use of the term “load lag” by Professor Spangler are explored. For the increase in pipe deflection following construction completion, the use of the term “time lag” should be used. A table of time lag values for use in estimating deflection is presented. The table is based on empirical data from field measurements.

KEYWORDS: flexible pipe, deflection, design, installation, time effects, soil–structure interaction, time lag

Introduction

A flexible pipe deflects due to load on the top of the pipe. For a given load, the deflection depends on the pipe stiffness and the stiffness of the soil placed at the side of the pipe. Equations to estimate the amount of deflection usually take the following form

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$$\text{Deflection} = \frac{\text{Load on pipe}}{\text{Pipe stiffness} + \text{Soil stiffness}}$$

There is also a time element. The initial deflection is the deflection that occurs as soon as the final backfill is placed over the pipe. The pipe typically continues to deflect, although little increase occurs after about 5 years. The long-term, or final, deflection is usually the value that is used for design purposes.

The term “deflection lag” is used in equations to estimate the final deflection. These equations appear in ASTM, AWWA, and ASCE manuals and standards. Typically, they are in the following format

$$\text{Deflection} = \text{Deflection lag} \times \frac{\text{Load on pipe}}{\text{Pipe stiffness} + \text{Soil stiffness}}$$

Professor M. G. Spangler from Iowa State first used the term deflection lag in his report in 1941 that first correlated flexible pipe behavior with the properties of the soil placed beside the pipe [1]. Unfortunately, the term has been mis-used and mis-applied in many of the equations used to predict long-term behavior of the pipe.

Spangler Deflection Lag

In the original Iowa formula, Spangler [1] presented a deflection lag factor that represents the increase in pipe deflection over time so that the final pipe deflection can be predicted.

What is overlooked is that the deflection lag factor that Spangler proposed is the increase in deflection *after the maximum load is reached on the pipe*. The maximum load on a pipe does not occur until 3–6 months after backfilling. Therefore, Spangler’s suggested value of 1.5 for deflection lag is not the increase in deflection from the day that the pipe is backfilled, as is generally assumed. His value of 1.5 is the typical increase in the pipe deflection from when the maximum load was reached on his test pipes (typically about 6 months) to his last reading several years later. Spangler measured the load on the pipe in his experiments and knew when the maximum load had been reached. In Spangler’s experiments, the final deflection is about twice the initial deflection (the deflection the day that backfilling was completed). This is illustrated in Fig. 1. In one of his experiments, the vertical deflection the day the backfilling was completed was 3.4 %. About 6 months later, the vertical deflection was 4.2 %. Thirteen years later, the vertical deflection was 6.2 %. His published deflection lag value of 1.5 was calculated by dividing the 13 year reading by the 6 month reading. In fact, the increase in deflection from the day of final backfilling to 13 years later was an increase from 3.4 % to 6.2 %, almost double.

According to Spangler in 1969 [2], a flexible pipe continues to deflect over time for two reasons:

- A. Increase in the soil load on the pipe.
- B. Compression and consolidation of the soil at the sides of the pipe.

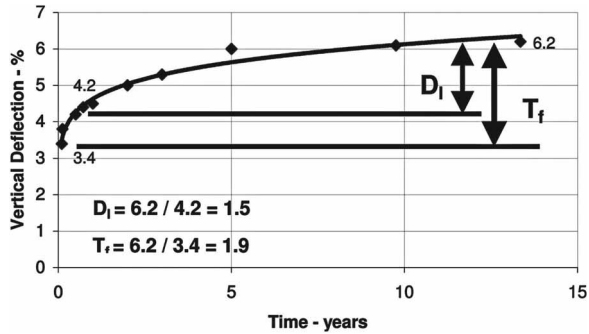


FIG. 1—Time lag versus deflection lag (Spangler test).

The Spangler deflection-lag factor covers reason B, but not reason A.

In essence, the Iowa formula, or Spangler equation, does not predict the initial deflection of a buried flexible pipe. It predicts the deflection when the maximum load is felt by the pipe. The maximum load occurs sometime about 3–6 months after final backfilling.

Spangler Load Lag

Spangler termed the increase from initial deflection to the deflection at the maximum load as “load lag.” Load lag results from the increase in load on the pipe as the backfill soil settles into place and into firm contact with the pipe. This is illustrated in Fig. 2.

The increase in load results from the soil settling into place against the pipe, less bridging of the soil over the pipe with time, less bridging of soil clods (the clod factor), soil structure redistribution due to water from rain, flood, or rise in ground water levels, and increased unit weight of the backfill soil due to additional water from rain or flood.

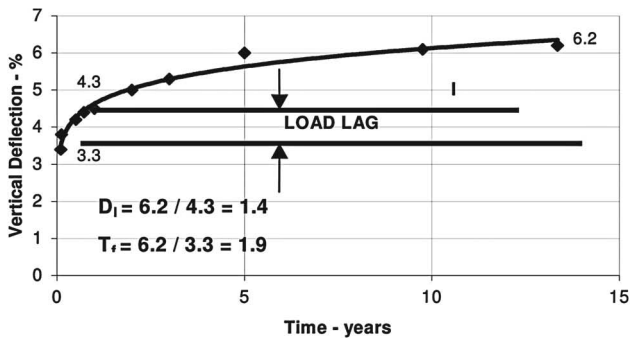


FIG. 2—Load lag (Spangler test).

Time Lag

In 1981, Howard proposed the term “time lag” to represent the increase from initial deflection over time [3]. The time lag factor, T_f , is the final deflection divided by the initial deflection. Although the flexible pipe continues to deflect, the increase after about 5 years is insignificant. Any measurement after 5 years can be considered the final deflection. As illustrated in Fig. 1, the time lag factor for Spangler’s experiment was about 2.

Howard also presented a table of time-lag values similar to the Reclamation E' table [3]. These values range from 1.5 to 3 and, similar to the E' values, vary according to the degree of compaction and soil type. The T_f values are based on field measurements from studies on buried pipe where the vertical deflection was monitored over time. These time-lag values are shown in Table 1. This table has been modified from the 1981 version to show crushed rock as only two compaction categories, uncompacted and compacted. This was done to parallel the 2006 changes made in the Reclamation table of E' values [4]. The change was made because there are no test methods available for measuring the in-place density of crushed rock material. An assessment can only be made for the effort made to compact crushed rock.

Note in the table that the time lag factors are lower for the lower densities. As Spangler first noted in 1969, the deflection increase over time is lesser for low density soils than high density soils [2]. For low stiffness soils beside the pipe, the soil compresses more initially and less over time as the pipe deflects. Conversely, the high stiffness soils resist the deflection initially and then compress over time. *Note that the higher values of time lag for the higher degrees of compaction do not mean high deflections.* For example, for a Poorly Graded Gravel (GP) compacted to 70 % Relative Density, the E' is 4000 lb/in.² (28 000 kPa). For a low stiffness pipe with 20 ft (6 m) of soil cover [120 lb/ft³ (1920 kg/m³)], the estimated initial average deflection is 0.6 %. The time lag factor from Table 1 is 2.5. The long term deflection would be 2.5×0.6 % or 1.5 %, well below an allowable deflection of 5 %.

If the embedment soils contain about 13 % or more fines, the embedment soil can soften when it becomes saturated, significantly if the soil density is moderate or less. Therefore, the time lag factor should be doubled if the ground water table will rise above the level of the top of the pipe. This can be avoided if the embedment soil is compacted to a density such that the water content of the soil with all voids filled with water is less than the liquid limit of the soil. This is typically about 95 % of standard Proctor density, but may be as high as 98 % standard Proctor. If the time lag becomes a concern in the soil–structure interaction design, then laboratory soil tests can be performed to evaluate the required density.

The field tests that were evaluated for Table 1 were mainly steel and filament wound fiberglass pipe. There were only a few polyvinyl chloride (PVC) and high density polyethylene pipe [8–11]. The plastic pipes are known to decrease in circumference due to burial confining pressure. Any circumferential shortening is not considered in Table 1.

The supporting information and data for Table 1 can be found in Ref. [5].

TABLE 1—Time-lag values (dimensionless).

Soil-Classification of Embedment	Degree of Compaction of Pipe Embedment		
	Dumped	Slight <85 % P <40 % RD	Moderate 85–95 % P 40 %–70 % RD
Highly compressible, plastic, or organic clays and silts, peat CH, MH, OH, OL, Pt	Do not use	Do not use	Do not use
<i>Clays and silts</i> With <30 % s and and/or gravel CL, ML, CL-ML			
Sandy or gravelly			
<i>Silts and clays</i> With 30 % or more fines sCL, gML, sCL, sML	1.5 ^a	2 ^a	2.5 ^a
<i>Sands and gravels</i> With 13 % or more fines GC, GM, SC, SM			
<i>Sands and gravels</i> With 12 % or less fines GW, GP, SW, SP	1.5	2	2.5
<i>Crushed rock</i> With less than 25 % passing 3/8 in. sieve and less than 12 % fines		2 ^b	3 ^c

Note:

1. Values only valid for pipe cover of 50 ft or less.
2. Values only valid when used with Reclamation equation and reclamation E' values.
3. Unified Soil Classification based on ASTM D2487 or D2488.
4. "Soil Classification of Embedment" also applies to dual symbol or borderline soils beginning with the symbol shown in column.
5. Fines are soil particles that pass a No. 200 (75 micrometer) sieve (clays and silts).
6. "P" is standard Proctor density (ASTM D698, AASHTO T 99) "RD" is relative density (ASTM D4253 and D4354)
7. All faces of "crushed rock" should be fractured.
8. Table not applicable for SP soils that are medium and fine sands dumped into place at the bulking moisture of the sand.

^aDouble time lag factor if embedment will become saturated.
^bUncompacted.
^cCompacted.

The time lag values were empirically derived using the Reclamation equation [6] for estimating flexible pipe deflection. The Reclamation equation calculates the vertical deflection using the prism load, reclamation E' values, and time lag values. The time lag values should only be used with this equation. However, the time lag values may be useful for roughly estimating final deflection using other methods for determining initial deflection and for roughly estimating the final deflection when the initial deflection has been measured

Marston Load Versus Prism Load

To calculate the soil load on a buried pipe, Spangler used the Marston load equation when he first published the Iowa formula [1]. However, when the backcalculated, empirical E' values were developed for the Reclamation E' table, a prism load was used in all the calculations [4]. The prism load is the weight of a column of soil over the pipe with a width of the pipe diameter, 1 ft long, and a height equal to the cover over the top of the pipe. As the E' values were developed using the prism load, the E' values *should only be used with a prism load*, not the Marston load. Use of the prism load is conservative because the Marston load results in less, or the same, load as the prism load resulting in less estimated deflection. All design guides, manuals, and trade association literature should use the prism load when using the Reclamation E' table.

Some references infer that the Marston load is the initial load on the pipe right after backfilling and the prism load is the final load on the pipe after several years. In these references, a deflection lag factor of 1.0 is recommended because the deflection is estimated using the prism load. This is technically incorrect because Marston and Spangler refer to the Marston load as the maximum load on a pipe and that it usually occurs 3–6 months after construction. The Marston load and the prism load are two separate methods of calculating the load on a pipe. While using the prism load to backcalculate E' values from initial deflection values to develop the Reclamation table of E' values is a simplification, it provides a straightforward way to calculate the load and then rely on time-lag values to estimate the final deflection of flexible pipe.

The author recommends using the Reclamation equation to estimate deflection of buried flexible pipe because it correctly includes the prism load, Reclamation E' values, and the time lag factors [6–12].

Summary and Conclusions

A flexible pipe deflects due to load on the top of the pipe. For a given load, the deflection depends on the pipe stiffness and the stiffness of the soil placed at the side of the pipe. The initial deflection is the deflection that occurs as soon as the final backfill is placed over the pipe. The pipe typically continues to deflect, although little increase occurs after about 5 years. The long-term, or final, deflection is usually the value that is used for design purposes.

The Iowa formula was published by Professor M. G. Spangler of Iowa State Univ. in 1941 to predict the deflection of buried flexible pipe. In the equation, he used the term deflection lag to describe the increase in flexible pipe deflection

after the maximum load was reached over the pipe. The maximum load on a pipe is generally attained after 3–6 months after construction. Professor Spangler used the term load lag to describe the increase in deflection as the load on the pipe increased and he used the term deflection lag to describe the increase in deflection after the maximum load was reached. Conclusions for use of time-dependent deflection increases are as follows:

1. Buried flexible pipe increase in deflection over time at a ratio of 1.5 to 3 of final to initial deflection. This increase should be accounted for in the various methods of estimating deflection.
2. The term time lag should be used to describe the increase in deflection from initial deflection to final deflection.
3. The term deflection lag should not be used to mean time lag. Deflection lag, as used by Spangler, has a totally different meaning.
4. Time lag values vary according to embedment soil type and density. Recommended values are presented in Table 1.
5. The Iowa formula, or Spangler equation, does not predict initial deflection. The method predicts the pipe deflection when the maximum load on the pipe is reached and this does not occur until about 3-6 months after backfilling. During this period, the pipe deflection increases as the load increases. Spangler referred to this as load lag.
6. After the maximum load is reached, the pipe continues to deflect due to the horizontal forces on the embedment soil. The side soil compresses similar to a building continuing to settle under the weight of the building.
7. The Reclamation equation correctly incorporates the prism load, E' values, and time lag values to estimate the deflection of a buried flexible pipe.

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TESTING AND FAILURE ANALYSIS

John M. Kurdziel¹

Stress Crack Protocol for Finished Product Testing of Corrugated High Density Polyethylene Pipe

ABSTRACT: The primary load bearing component of a dual wall corrugated high density polyethylene pipe is the annular corrugated wall. It is this area that necessitates engineering evaluation with regard to the maximum permissible strains and the associated stress crack resistance of this thermoplastic material. This paper presents the protocol for evaluating the stresses in these critical structural members with new testing methods for determining the long-term stress capacity or service life for these members. This test method provides an alternate long-term stress crack evaluation protocol that will allow the user to design piping systems for a desired service life with more accuracy than simply specifying hydrostatic design basis rated resins.

KEYWORDS: stress cracking, HDPE, test protocol, corrugated polyethylene pipe, service life, fracture mechanics, notch ring compression test, stress crack resistance

Introduction

The service life of dual wall corrugated high density polyethylene (HDPE) pipe has historically been assessed based on stress cracking at the inner liner and corrugation wall interface. Although this circumferential cracking is easy to observe and assess, it does not represent a critical structural component of the pipe. The fact that one could remove the entire inner liner of a dual wall corrugated HDPE pipe and not significantly influence its structural performance is evidence of the fallacy of basing a service life on such a non-critical component. The highest tensile stress locations of buried corrugated HDPE pipe occur in the outer most fiber of the corrugation, where the bending strains are the

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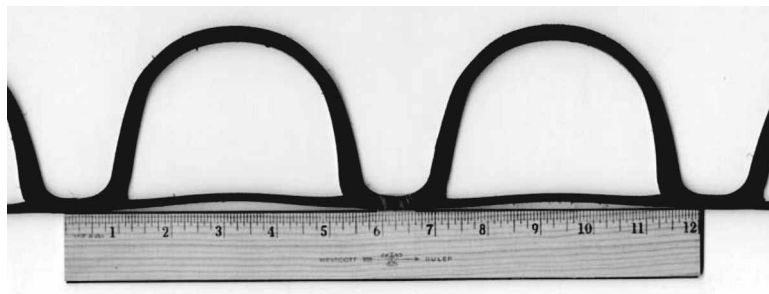


FIG. 1—Corrugation HDPE pipe profile.

greatest. Compression related strains do not result in any long-term stress cracking, so one must concentrate on tensile strains. These tensile bending strains are associated with deflection, which is a key AASHTO design parameter. It makes sense, therefore, to concentrate our efforts on determining the stress crack resistance of these corrugation members rather than on a non-structural component of the pipe, such as the liner.

Evaluation of Stresses in HDPE Corrugate Pipe Profiles

Corrugated HDPE pipe is a profile wall pipe with three distinct cross-sectional components (Fig. 1): The corrugation, valley, and liner. The corrugation consists of a crest, web, period, and depth or height, but for discussions in this paper, they will be discussed as a monolithic unit. The wall thicknesses and geometric configurations of each of these primary components vary significantly from one another. Based on the geometric intersections of these three components and the large differential in cross-sectional area, an inherent stress riser, point A, is created at the intersection of inner wall and exterior corrugation (Fig. 1). Due to these factors, under high stress conditions, circumferential cracking may occur at this location. It is this cracking that has prompted much of the concerns pertaining to the service life of corrugated HDPE pipe.

If one examines the impact of circumferential cracking at the liner-corrugation interface, it is clearly not a principal design concern as it does not result in any structural distress in the pipe. This cracking does not even increase the possibility of infiltration or exfiltration in the pipeline as the circumferential cracking at this interface only opens a path to the inside of the corrugation. Since the cracking also does not typically result in any significant offsets that may affect hydraulic performance, these cracks are merely cosmetic and may be repaired in place with no long-term implications to the pipe's performance.

The principal structural components must therefore be identified and the stress cracking assessed and service life determined in these members. With the elimination of the inner liner, the two remaining members are the corrugation and valley. The valley is by far the thicker of the two members as it is the bonded combination of the inner wall and corrugation extrusions. This member also

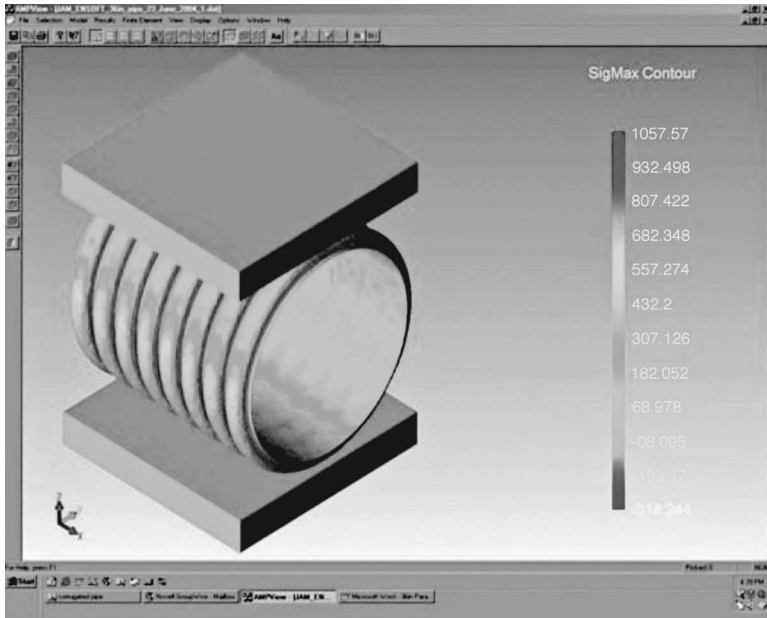


FIG. 2—High stress locations associated with parallel plate loading.

experiences a lower level of strain than predicted by the AASHTO design criteria [1], and due to external load distribution and profile geometry in a typical corrugated HDPE pipe, this member experiences predominately compression not bending strains.

In comparison, the corrugation is a much more complex member, subject to relatively high bending strains depending on the height of the corrugation. In any event, this member would have the thinner of the two walls with respect to the valley, and its extreme fiber would be the furthest from the centroid of the wall. Both of these factors indicate that this member would be the most likely to have the highest bending strains and, therefore, the greatest possibility to have stress crack issues that could affect the structural durability of a corrugated HDPE pipe.

Figure 2 represents a finite element model of the parallel plate test loading condition for corrugated HDPE pipe. The highest stresses occur circumferentially on the inside surface of the crown and invert, and the external outer most fibers at the springline as a result of bending. It is this latter area, which is of most interest in assessing the stress crack resistance of the performance of the key structural members in corrugated HDPE pipe.

This protocol concentrated on the principal structural member, the corrugation, which has the highest tensile strain from deflection and greatest potential for point stresses from rock impingement [2].

For annular corrugated HDPE pipe, the means for estimating service life has historically relied on conducting laboratory testing small specimens that are cut from the pipe wall or samples made from resin pellets or reground

sections taken from the pipe. These tests are accelerated via temperature and applied stress and may include a chemical accelerant, such as an Igepal solution.

Full Scale Pipe Testing

By design, thermoplastic pipe is expected to deflect, with most specifications, including those for solid wall HDPE pressure pipe and corrugated HDPE pipe, limiting diametric deformation to 5 %. Developing a model to simulate field deflection from earth loading was extremely complicated due to the impact of the restraining forces associated with soil pressures. It was found, however, that the maximum bending stresses at the outer fibers of the pipe wall would develop if the pipe was simply deformed between two rigid members as is the case with a parallel plate test.

A further parallel plate finite element (FE) analysis [3] indicated that the cross-section of a corrugation was deformed excessively when a ring of pipe was compressed between smooth plates. In addition, the point of load application shifted as the ring deformed, thus significantly changing the stress distribution within the ring. The results of the FE modeling and experiments suggest that the loads to be applied in the tests should be very low as these loads will keep the minimal stresses and strains within a reasonable range relative to the typical design allowable for HDPE material. Buckling or wall crushing of the webs of the corrugation will be unlikely because the loads are so low.

The optimal test method should, therefore, be a modification of the Palermo/Kurdziel test method that concentrates the highest stress at the spring-line of the pipe without changing or altering the load distribution. To quantify the stress crack resistance of a critical element, a further analysis was conducted and approached through the use of fracture mechanics.

Fracture Mechanics Evaluation

The study [3] undertaken with Battelle by Advanced Drainage Systems (ADS) provided a more quantitative means for determining the magnitude of the applied load and a protocol for a test method for evaluating the stress crack resistance of annular corrugated HDPE pipe. This work concentrated on characterizing the crack growth rate in terms of stresses acting at a crack tip and equating them to the degradation of service life.

With fracture mechanics, the behavior of a pipe or specimen containing a sharp crack can be analyzed. The sharp crack has an infinite stress concentration and therefore cannot be evaluated with common structural analysis techniques. A benefit of using a specimen with a crack is that the crack focuses the highest stresses and strains onto a specific volume of material. It is more likely that the long-term durability of the pipe can be assessed if this volume of material resides in a critical area of the pipe. In addition, focusing these stresses and strains makes it possible to more aggressively accelerate the test without necessarily changing the failure mode. Moreover, without the crack, it is likely that the specimen would have a considerably longer service life, thus pre-damaging

the specimen by introducing the sharp crack should produce a conservative, lower bound on the service life determined by the test.

Fracture mechanics provides a formulation for extrapolating the behavior of the cracked laboratory specimen to the pipe in the field. This is done by measuring the asymptotic stress field near the tip of a sharp crack. This methodology is a function of the structural geometry, the size, orientation, and location of the crack, and the magnitude and nature of the applied load. When the governing equations of linear elastic fracture mechanics apply, the response of the crack in the pipe will be the same as the crack in the laboratory specimen when both cracks exhibit the same asymptotic stress field. Essentially, the pipe will yield when the stress in the pipe equals the stress that cased the laboratory specimen to yield.

The main difficulties related to this methodology are measuring the crack length and impacts of multiple crack tips. Of these two issues, multiple crack tips is the more difficult to evaluate as an average crack growth rate can be accurately determined by simply taking measurements at a particular start and stop time of a test. The consistency and depth of a scalpel cut can significantly impact the results of a test and consequently yield much different results from one test laboratory to another and even among one laboratories own data. For this reason, a reproducible test protocol for making the scalpel cut is critical. This protocol must include the depth, width, location of the scalpel cut and the quality of crack tip, which is heavily influenced by the number of cuts made by a particular scalpel. A scalpel that has been dulled by even a few cuts may blunt the edge of the cut and result in differential crack growth or the development of multiple fracture cracks, which would dissipate the loading. Either of these effects would essentially invalidate the capability of obtaining reproducible test results.

Prototype Test Method

The prototype test method is the Notched Ring Compression Test (Fig. 3) that is similar to the ring compression test or the Kurdziel/Palermo method with the exception that a notch is made at both the inside crown and invert to reduce the affect of the stiffness at the contact points. This test has the same benefits of the previously discussed test methods without any of the disadvantages. In addition, the section may actually permit accelerated testing without a crack initiation. The one additional disadvantage is the requirement to cut vertically aligned notches.

Notched Ring Compression Test

The Notched Ring Compression Test is a creep growth test conducted in a load control test frame. A saw cut starter crack is cut through the corrugation crown with the end sharpened with a scalpel cut. This procedure eliminates a considerable amount of variability by not placing the scalpel cut on a curved corrugation crown where reproducibility would be difficult to obtain. Slow crack growth is

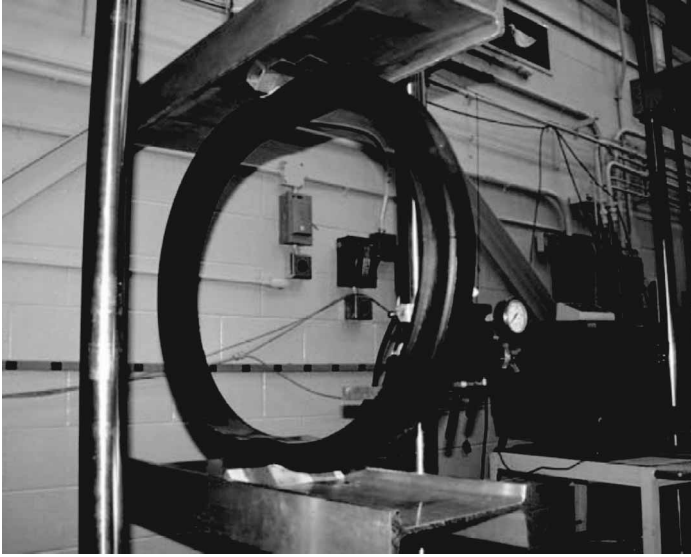


FIG. 3—*Notched ring compression test.*

easier to ascertain by examining the web of the corrugation (Fig. 4) to determine the crack phases associated with loading.

A standard 24-in. ADS corrugated HDPE pipe was used for determining the initial test protocol. The proposed test protocol for 24-in. pipe commences with a ramp up of load to 39 lb in 52 s to apply the initial stress. It is held constant at 39 lb for 147 h to determine the crack propagation and then displaced to its full stroke of 10-in. in 56 s to failure. The load and stroke are recorded throughout the test (Fig. 5).



FIG. 4—*Crack plane.*

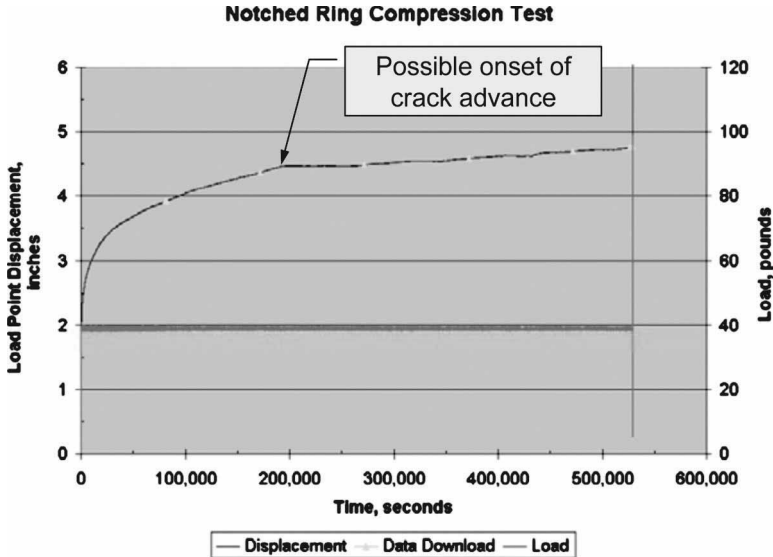


FIG. 5—Notch ring compression test results.

As can be observed in Fig. 4, in the slow crack growth regime, there was some degree of crack tunneling through the center of the wall, which made direct measurement of the crack length from the surface difficult without a correction factor or a direct cross-sectional measurement. The creep crack growth, however, was simulated accurately through the associated finite element analysis that was conducted on this test. The compliance as indicated by the slope of the load point displacement versus time plot was observed to decrease with increasing crack size, an indication that the test introduces additional variables, possibly due to the benefit of geometric dispersion, as the crack propagates to its limits.

Test Apparatus

With the development of any test protocol, it is as equally important to develop a simple and inexpensive means of testing a product as it is to determine its performance. A number of the test concepts were eliminated due to the complexity and cost of the test apparatus or specimen preparation. The test frame developed for conducting the Notch Ring Compression Test is both simple and inexpensive (Fig. 6).

This test apparatus is designed and recommend for use on 12–60-in. corrugated HDPE pipe.

Test Ring Preparation

The following steps are used to create a sample test ring.

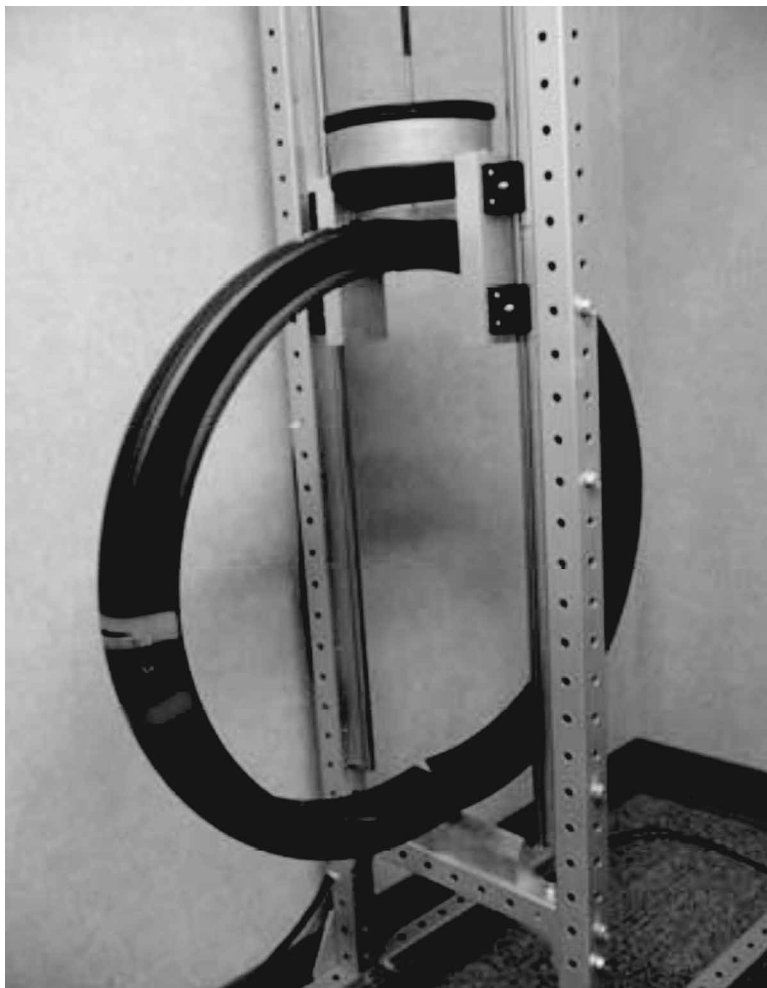


FIG. 6—*Test apparatus.*

- (1) A single corrugation is cut from a stick of pipe. The sample must be cut such that one full period of corrugation is represented in the test specimen. The sample should be symmetrically cut in adjacent valleys.
- (2) The ring should be marked with an "A" side and "B" side.
- (3) The ring is then marked at 90° intervals around the circumference of the ring. Positions should be marked as 0°, 90°, 180°, and 270° progressing clockwise around the A side of the test ring (see Fig. 7).
- (4) Flip test ring over and mark the opposite side at 90° intervals around the circumference of the ring. Positions should be marked as 0°, 90°, 180°, and 270° progressing counter clockwise around the B side of the test ring.

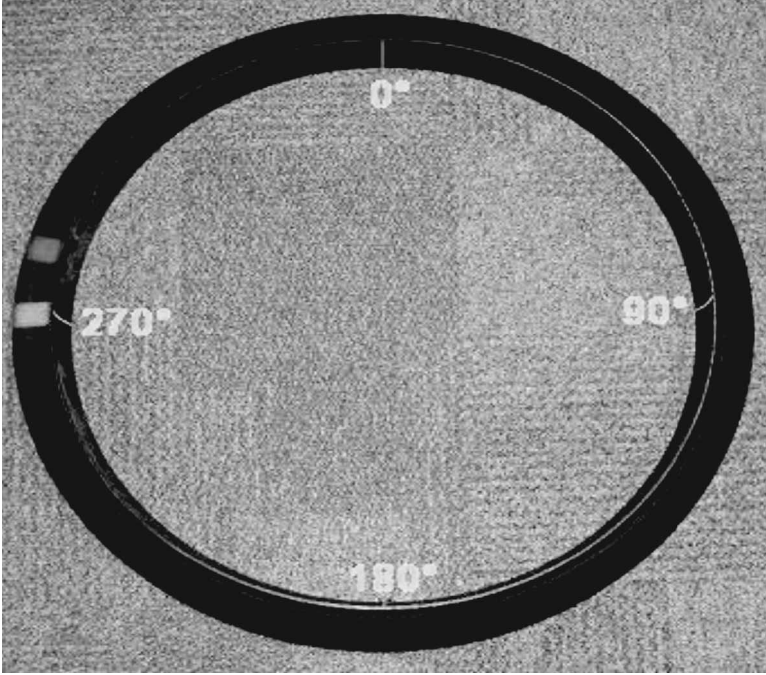


FIG. 7—*Test ring.*

- (5) Centered in the crown of the corrugation at the 0° and 180° positions, drill a $5/16$ -in. hole through the crown of the corrugation (see Fig. 8). These holes will be used to position guide pins from the test apparatus such that the test specimen will not dislodge from the test apparatus during loading.

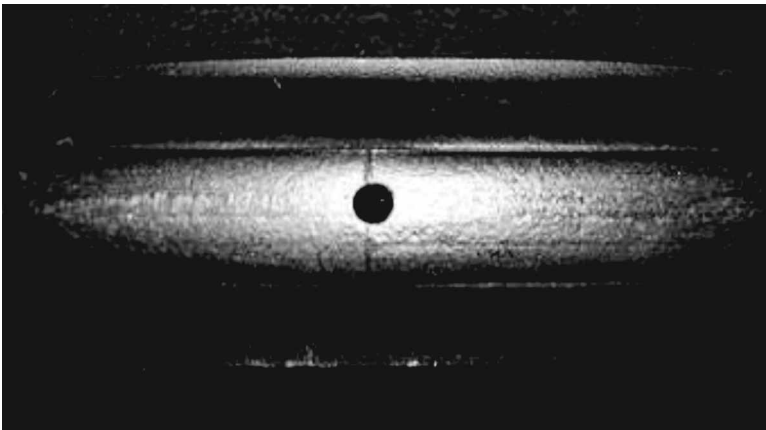
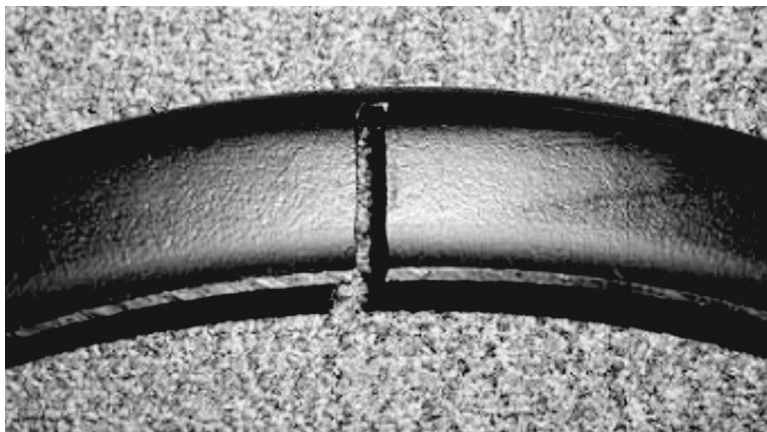


FIG. 8—*Drilled 5/16-in. hole.*

FIG. 9—*Relief cut.*

- (6) A 1/4-in. wide relief cut is then made through the liner and sidewall of the corrugation at both the 0° and 180° positions (see Fig. 9). The relief cut shall be perpendicular to the corrugation crown at both the 0° and 180° positions and progress completely through the liner and sidewall such that a plastic hinge is formed from the corrugation crown.
- (7) Using a band saw, a thin slot is cut through the corrugation crown and into the corrugation sidewall. Slots shall be created at both the 90° and 270° positions (see Fig. 10). The slot depth is specified as a percentage of the corrugation height.
- (8) A razor blade is then used to increase the slot depth by 1/8-in. (see Fig. 11). This produces a sharp edge crack initiation at the tip of the notch in the corrugation at the bottom of each slot. When complete, razor

FIG. 10—*Slot.*

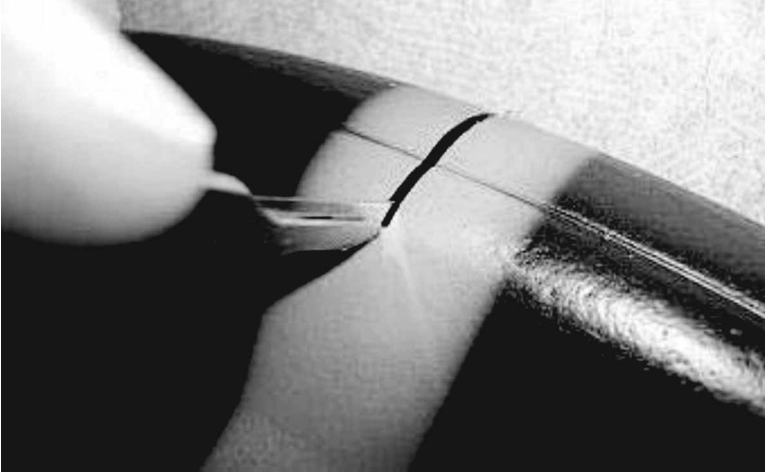


FIG. 11—*Razor notch.*

blade notches shall be placed at the 0°_A , 0°_B , 270°_A , and 270°_B locations and the locations labeled accordingly (Fig. 12).

Test Execution

Once the test ring is prepared, the test setup should progress as follows.

- (1) Based on the test ring to be used, prepare the top anvil with the proper test weight. This selection is based on the pipe diameter and profile geometry, which will initiate a stress crack.
- (2) Place the 5/16-in. hole located at the 180° position of the test ring over the bottom anvil guide pin (see Fig. 13).
- (3) Slowly lower the top anvil so that the top anvil guide pin is inserted into the hole located at the 0° position on the test ring (see Fig. 14). While the guide pins should be inserted into the test ring, weight should not be applied to the test section at this time.
- (4) Initiate a data recorder to take a deflection reading taken every 5 min. The data recorder should record the time and deflection magnitude.
- (5) Slowly lower the test weight (top anvil) onto the test ring (see Fig. 15). Weight should be applied at a slow, smooth, and constant rate.
- (6) Once the top anvil is fully supported by the test ring, fully release the test weight making sure not to bump or agitate the test setup. Abrupt loading, bumps, or non-smooth load application may result in a blunt crack tip, pre-mature crack growth, or tearing at the crack tip.
- (7) Test shall progress such that the test ring has reached 40 % deflection based on the test ring inside diameter.
- (8) Gently remove test weight from test specimen. Weight should be removed with a smooth and constant motion. Precaution should be taken as to not abruptly unload or bump sample.
- (9) Remove test specimen from upper and lower anvil guide pins.

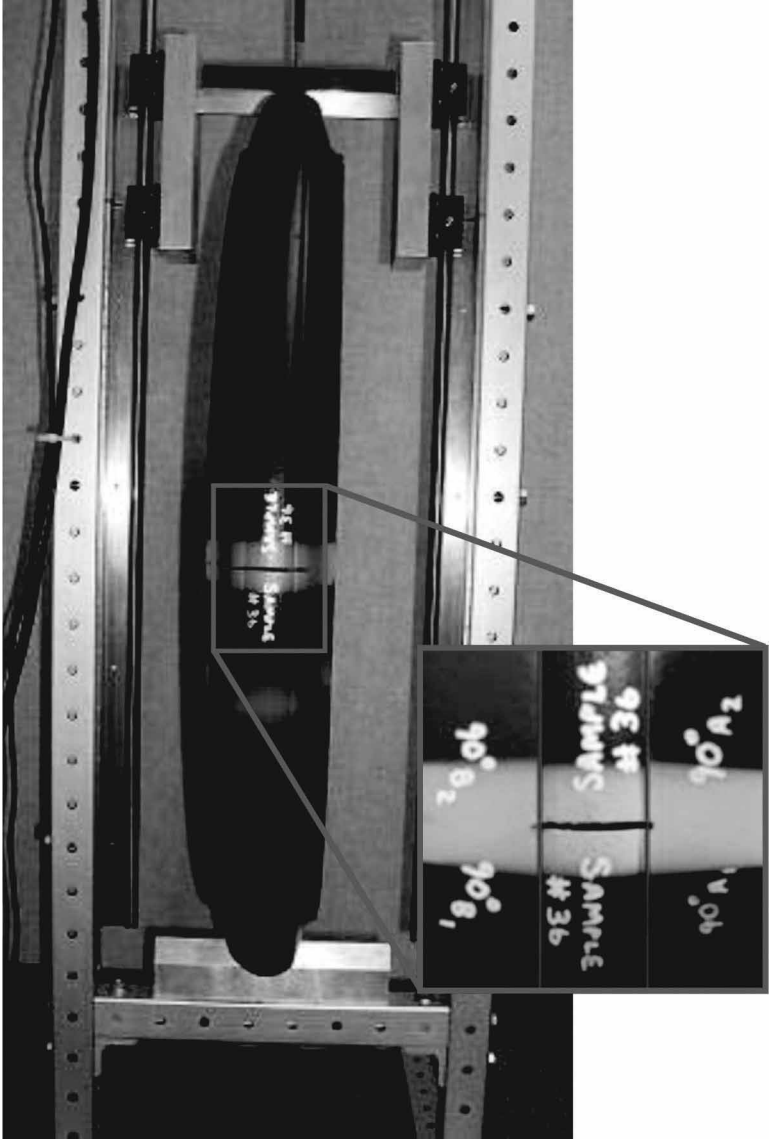


FIG. 12—Labeling the slots.

Data Collection

In order to determine crack growth rate, the length of ductile crack growth must be measured. Due to tunneling of the crack tip and masking at the crack surface, the sample must be dissected at each crack location. Once dissected, the crack length can be measured using a microscope and micrometer. Upon completion of testing, data should be collected from the test specimen as

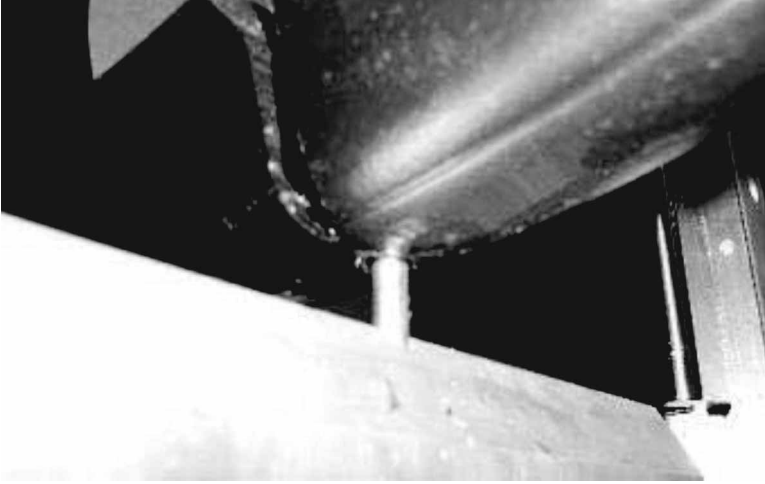


FIG. 13—*Bottom anvil pin.*

follows. Note that once testing is complete, precaution must be taken as to not bend specimen.

- (1) In order to prepare crack section for analysis, the sample length must be reduced at both the 90° and 270° locations. Cut the 90° and 270° locations free from the test ring. Each section should be roughly cut 8-in. in length (see Fig. 16).
- (2) Place a 1/4-in. wide relief cut in each of the two test specimens (90° and 270° sample). Relief cut should extend through the liner and up the



FIG. 14—*Top anvil pin.*

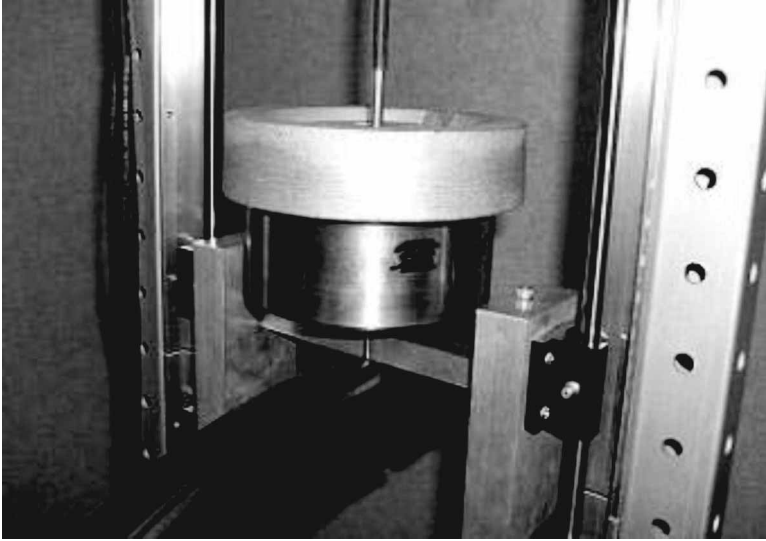


FIG. 15—Test weight.

sidewall to a distance no closer than 1/4-in. from assumed crack tip (see Fig. 17).

- (3) Condition test specimen at 72°F for not less than 1 h.
- (4) The crack growth mechanism during testing should be ductile tearing. In order to dissect the sample without adversely affecting the crack growth from the testing, a brittle break of sample is necessary to expose the true crack length. Brittle breaking is best achieved by freezing the



FIG. 16—Crack section.



FIG. 17—*Relief cut.*



FIG. 18—*Nitrogen freezing.*

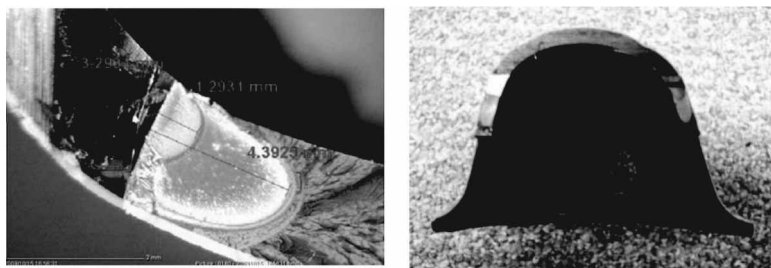


FIG. 19—Crack surfaces.

test specimen. To freeze test specimen, fully submerge specimen in liquid nitrogen for not less than 5 min (see Fig. 18).

- (5) Remove test specimen from the liquid nitrogen, and with a quick bending motion, break open the crack so that the crack surface is now fully exposed (see Fig. 19). Completion of this step will result in four exposed crack surfaces per test location (90°_{A-1} , 90°_{A-2} , 90°_{B-1} , 90°_{B-2} , 270°_{A-1} , 270°_{A-2} , 270°_{B-1} , and 270°_{B-2}).
- (6) Crack length of each crack surface should be measured from initiation to crack tip and recorded. Measurement should be taken under a microscope with 40-1 magnification and suitable micrometer or image analysis software (Fig. 19).
- (7) Using the time/deflection data collected earlier, time for crack growth should be recorded by subtracting the crack initiation time from the total test time. Crack initiation time can be determined by analyzing the shape of the time/deflection curve and locating the position of a point where deflection rate (crack growth rate) has normalized or reached a steady rate (see Fig. 5). The length and time or the rate of growth of the crack from the point after the crack initiation time will yield the relative stress crack resistance of the pipe when extrapolated over the life of the project.

Summary

The development of a test protocol and apparatus for evaluating stress crack resistance of finish, full size product via stress crack resistance is critical to determine the service life of annular corrugated HDPE pipe. The Notch Ring Compression Test provides a new proposed method for determining service life. The validation of the test protocol is still necessary to confirm that it is a reproducible test method for various profiles. The protocol presented in this paper, however, presents a detailed laboratory test for determining when and how to measure the initiation of the stress crack resistance of a specific diameter pipe profile under an applied load.

The final aspects of this research will be the correlation of the applied stresses in a field installed pipe to those of a laboratory sample. With this test protocol, it is the intent to not only be able to determine the stress crack

resistance of a field installed pipe but also project the service life of a pipe based on these long-term stresses.

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Guided Side-Bend: An Alternative Qualification Method for Butt Fusion Joining of Polyethylene Pipe and Fittings

ABSTRACT: Historically, the procedure for butt fusion joining of polyethylene pipe has been qualified on the basis of two fundamental tests that are widely recognized within the industry. These are the tensile-impact (T-I) test method found in ASTM F2634-07 and the reverse-bend (RB) or bend-back test as described in ASTM F2620-06. Both of these test methods have been extensively utilized within the industry, and both have some specific limitations as it relates to qualification of butt fusion joining conducted in the field. ASTM F2634-07 is a highly sophisticated testing method involving precision instrumented laboratory equipment. This makes it impractical for an on-site qualification for butt fusion procedures in the field. The bend-back test, on the other hand, lends itself to field implementation, but as the method is applied to a larger diameter, heavy-walled pipe, the practicality of performing the test and safety considerations become a limiting factor. This paper will examine an alternative test method referred to as guided side-bend or full-face testing that appears very promising as an alternative to the RB test as described in ASTM F2620-06. The test method will be fully described, and data will be presented comparing T-I results and traditional bend-back results to those obtained using the side-bend test method. The discussion of the test results and safety considerations of the proposed test method will be addressed, and conclusions will be drawn regarding its future applicability.

KEYWORDS: PE, pipe, fusion, joining, testing, qualification

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Introduction

Research continues on the development of non-destructive testing methods to assess the quality of butt fusion joints made on polyethylene (PE) pipe. In the interim, the industry continues to use various destructive methods to qualify the procedure used to make these joints, combined with visual inspection and retention of data pertaining to the fusion joining parameters by which the joints are made as a basis for quality assessment. Principal among the fusion procedure qualification methods are (a) the bend-back or reverse-bend (RB) test per ASTM F2620-06 (herein referred to as ASTM F2620-06) and (b) the tensile-impact (T-I) test per ASTM F2634-07 (herein referred to as ASTM F2634-07). [1,2]

This paper examines each of these two qualification methods and discusses the advantages and disadvantages of each. From this discussion, it is concluded that neither method represents a practical field-oriented method for the qualification of the butt fusion joining procedure for large diameter, heavy-walled (> 1.00 in.) PE pipe.

Extensive research have been conducted into this area and determined that an alternate test methodology, known as guided side-bend (GSB) testing or full-face testing, may represent a more practical, safer, and more discerning approach to the qualification of the butt fusion joining procedure. In the discussion that follows, the GSB test will be summarized and test results will be presented relative to this method of butt fusion qualification as compared to the more traditional RB or T-I testing method.

It should be recognized that test results obtained to date suggest that the GSB test represents a viable alternate method for butt fusion qualification particularly for large diameter, heavy-walled (> 1.00 in.) PE pipe. The investigation of this test method is ongoing at the time of this writing, and as such, more test results are being developed for correlation and analysis. From this effort, it is hoped that this research may form the basis for a much safer, more accurate, and field-friendly ASTM test method to be used in the qualification of the butt fusion joining method for PE pipe.

The RB Test

The RB test is described within Appendix X4 of ASTM F2620. Variants of the same test method are also found in ASTM D2513, ANSI/AWWA C901, and ANSI/AWWA C906 [3–5]. The RB test has served the PE pipe industry well for many years. The test is relatively simple to understand and conduct and, with some degree of caution, lends itself well to field qualification of butt fusion joining procedures.

In theory, the test procedure is as follows. A strap is cut from a butt fusion joint along the axis of the pipe. The strap cut from the pipe containing the fusion joint typically has a width of $1\frac{1}{2}$ times the pipe wall thickness and a length of 15 times the pipe wall thickness as measured from the centerline of the fusion joint and extending along the axis of the pipe in each direction from the fusion joint (see Fig. 1).

The strap is then placed in a vise or other suitable fixture to essentially allow for reverse curvature of the pipe back onto itself, thereby placing a maximum

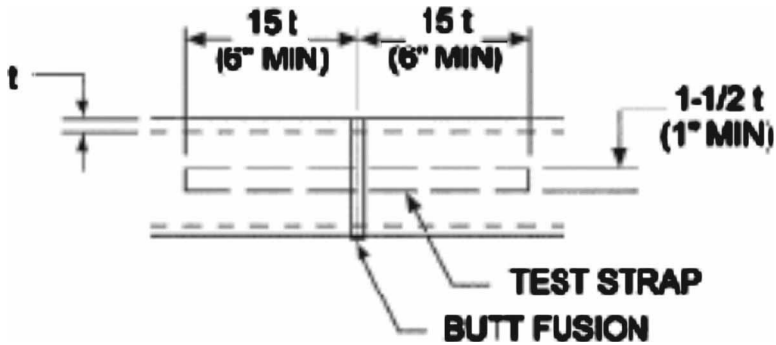


FIG. 1—RB test strap per ASTM F2620-06.

strain on the inside surface of the pipe at the region of the butt fusion joint. Alternatively, the strap may be bent by flexure around a mandrel of fixed diameter as shown in Fig. 2. The acceptance or rejection of the fusion joining procedure is typically determined by cracking or complete failure of the test specimen in the region of the interface of the butt fusion joint.

In the case of ASTM D2513, ANSI/AWWA C901, and ANSI/AWWA C906, the RB test is actually used for quality assurance testing of the extruded pipe and less so as a qualification of the butt fusion procedure. For specifics regarding the use and interpretation of test results in RB testing within these standards, the reader is referred to the standard of interest.



FIG. 2—RB test laboratory demonstration.

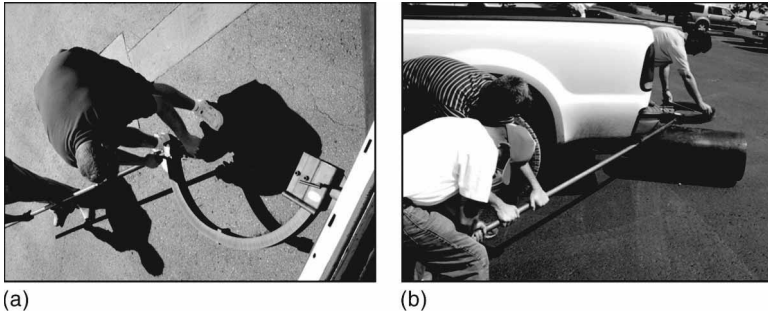


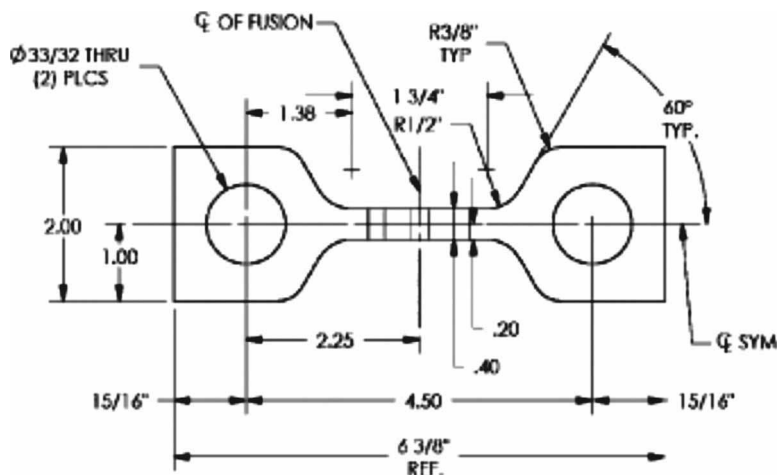
FIG. 3—RB tests conducted in the field on heavy-walled PE pipe.

As noted, the RB test has been used for many years within the PE pipe industry. The original test was developed as a means to qualify the butt fusion joining procedure for smaller diameters of PE pipe with correspondingly thinner walls (generally less than 1.00 in.) in applications such as gas distribution. In this regard, the RB test is still a very useful and field-friendly test method. However, as the industry at large has evolved into much larger diameters and much heavier walls (> 1.00 in.), shortcomings in the test itself have become more apparent as it relates to field friendliness and overall safety considerations. These two aspects are independent but highly inter-related.

From the perspective of field adaptation of RB testing, it has been found to be more difficult to perform in a manner that is consistent with laboratory conditions. First, a clamping mechanism or vise must be attached to a stable piece of equipment at the job site. Second, the gripping and deflection of the sample must be performed very carefully to ensure that the butt fusion joint is centered in the area of the maximum concentrated strain. This same gripping and deflection requirement must be performed in such a way as to ensure that nothing slips or loosens during the test procedure.

The second deficiency of the RB test as it is conducted in the field relates to increased concern over safety considerations when the test is conducted on butt fusion specimens taken from large diameter and/or very heavy-walled (> 1.00 in.) PE pipe. Tremendous energy is stored in a deflected PE pipe sample, and if anything slips or comes loose in the process of conducting the RB test, serious injury can result. Figure 3(a), 3(b) shows specific devices that have been used in the field to conduct RB testing. As larger diameters and heavier walls of pipe become more commonplace, safety concerns associated with the RB test and using devices such as those shown in Fig. 3(a), 3(b) become more and more significant.

Finally, proponents of the RB test argue that it does effectively test the butt fusion joint by placing maximum strain on the outer fiber of the test specimen. Detractors of the test method, on the other hand, suggest that it is more important to test the full cross section of the butt fusion joint in order to more effectively evaluate the integrity of the butt fusion joint.

FIG. 4—*T-I coupon per ASTM F2634-07.*

Tensile-Impact Testing

The T-I standard test method in ASTM F2634-07 has been used by some companies as a qualification method for the butt fusion process used to join PE pipe. In this test method, a specimen is cut from across the butt fusion joint to be qualified. The final specimen's dimension and shape are shown in Fig. 4. The reader should note that for the pipe wall thicknesses in excess of 2.60 in., the samples thickness must first be reduced to something less than 2.60 in. and the test coupon machined on high quality machining equipment. The specimen is then placed in a properly calibrated and instrumented tensile test apparatus capable of producing controlled, rapid displacement of the specimen in the axial direction (see Fig. 5). The speed of displacement is dependent on wall thickness of the test specimen as specified in the test method.

The T-I test apparatus then applies the specified rate of displacement (strain), and various test parameters are recorded as stipulated in the test method. The test method provides both qualitative and quantitative test results that are used to assess the integrity of the butt fusion joint under evaluation. The acceptance or rejection of the joint integrity is based on both a visual inspection of the test specimen upon completion of the test and a comparative evaluation of the tensile rupture curve generated during the test procedure. Figures 6 and 7 illustrate the differences observed in a ductile failure of a butt fusion joint and a brittle failure of a butt fusion joint when tested in accordance with ASTM F2634-07. For more details regarding the T-I test method, the reader is referred to ASTM F2634-07.

ASTM F2634-07 has become an extremely important test method within the PE pipe industry as it can be used not only for both qualification of the butt fusion procedure but also for research and development of butt fusion joining parameters in general. While ASTM F2634-07 is an efficient test method providing accurate and reproducible test results, it does have its limitations with

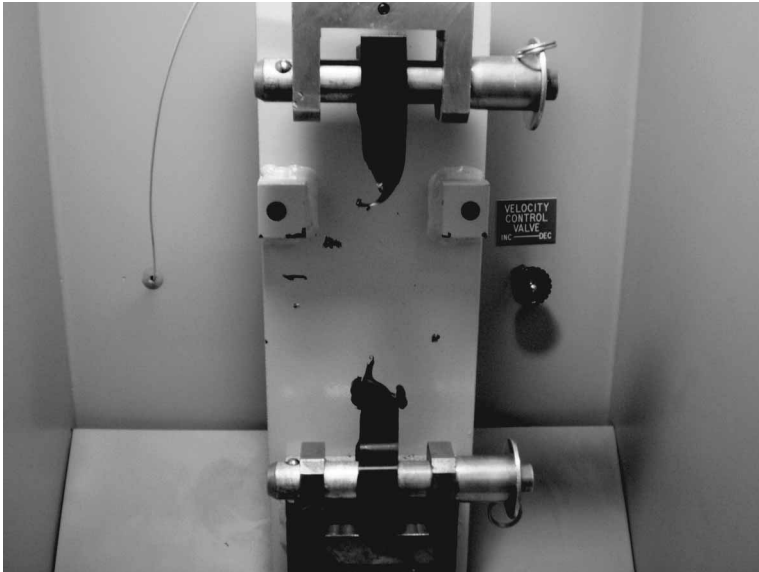


FIG. 5—*T-I testing apparatus.*

regard to butt fusion procedure qualification. Principal among these is that it is a laboratory test. This single fact makes it impractical as a field-friendly method for the qualification of fusion joining procedures. Specimens must be taken from a butt fusion joint made at the job site and then physically sent to a shop for machining and then onto a laboratory for testing and evaluation. The intermediate step (the machine shop) can be both time-consuming and costly as it often involves a third party and precision, computerized, numerical controlled machining equipment. Test results are then communicated back to the job site, but the time required to do so may pose significant delays. A final consideration is the overall cost of the laboratory equipment, its calibration, and maintenance. Research has recently been initiated to investigate the feasibility of modifying the ASTM F2634-07 test method and/or equipment to create a more “field-friendly” approach.

The Guided Side-Bend Test

Given the disparity apparent between the two prevailing test methods widely utilized within the North American HDPE pipe industry, it should not come as a surprise that alternative methods of heat fusion joining qualification are being sought. To this end, the authors have conducted extensive research into the GSB test method as a means by which to establish a safe and reliable field-friendly method for qualifying heat fusion joints made to an established joining procedure.

The GSB test as developed here involves the preparation of a fusion test specimen that is subsequently placed in a cost-effective, field-adaptable test

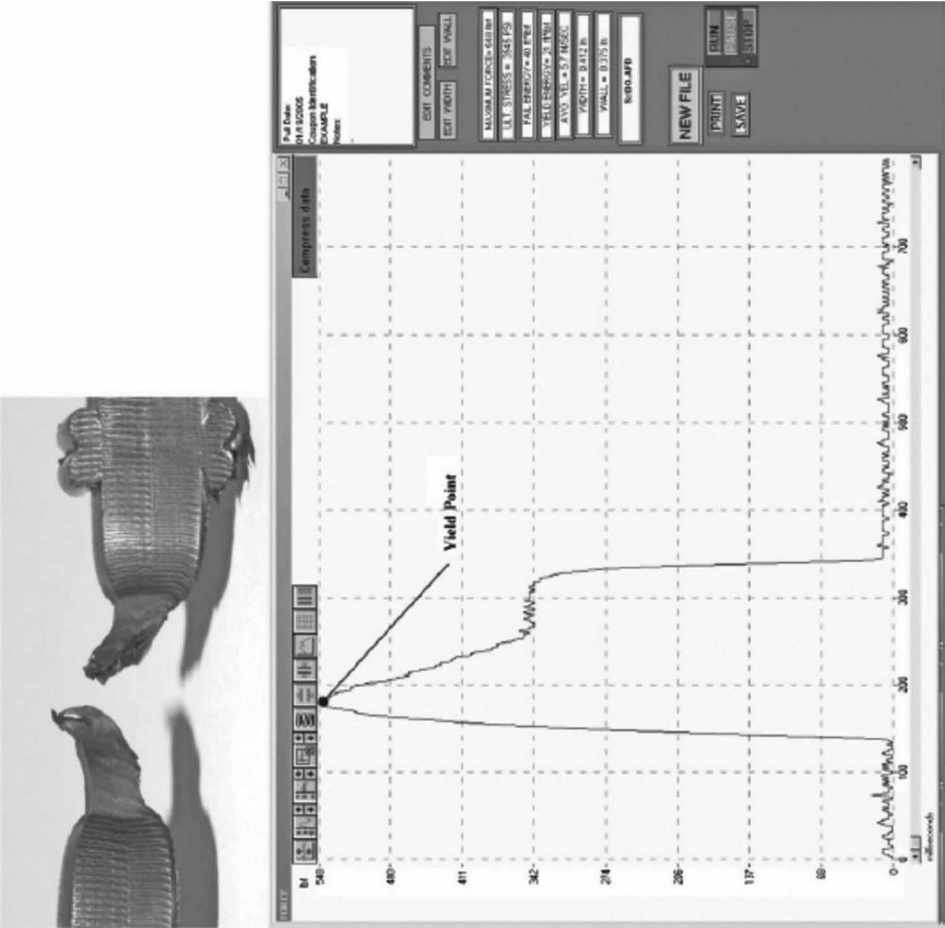


FIG. 6—Test output and example specimen of ductile rupture per ASTM F2634-07.

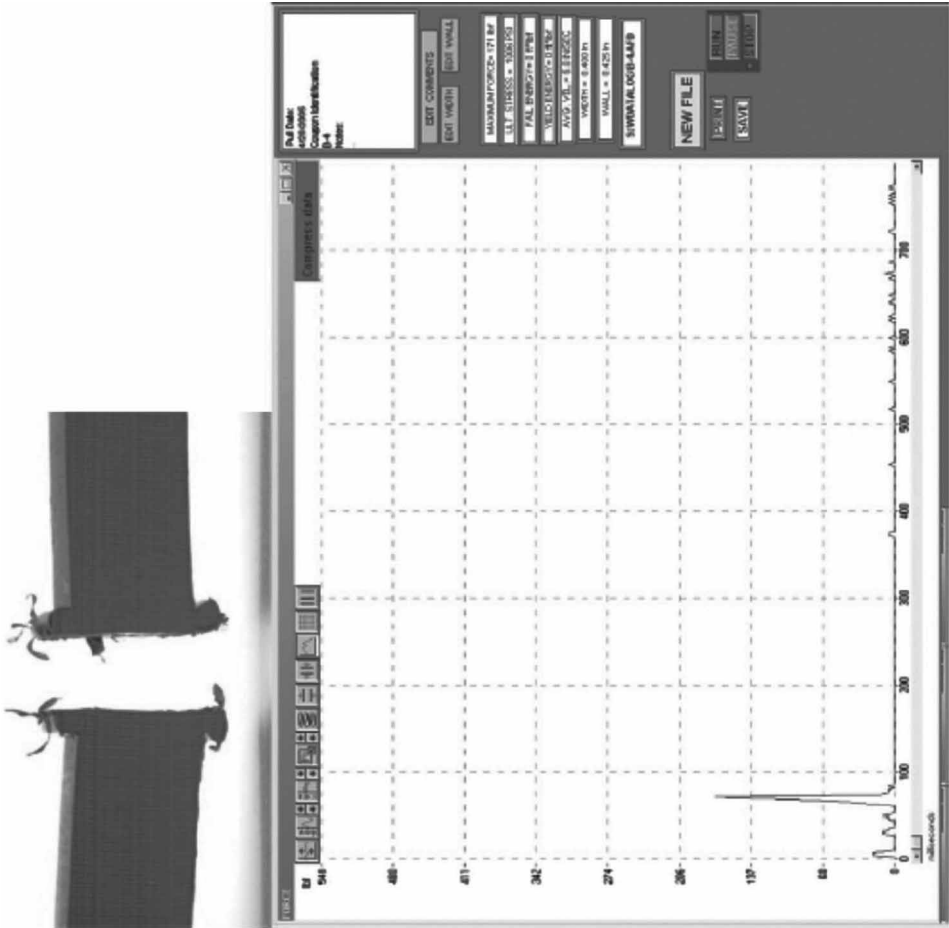


FIG. 7—Test output and example specimen of brittle rupture per ASTM F2634-07.



FIG. 8—GSB test specimen.

apparatus and that is then subsequently bent in a controlled manner along the entire width (or face) of a heat fusion joint.

The GSB test is relatively new to the U.S. HDPE pipe industry. However, on an international level, it has been used quite successfully for assessing the integrity of heat fusion joints. A standard test method does exist as BS EN 12814. 6 Briefly, the GSB test method as researched here can be summarized as follows.

Sample Preparation

Sample preparation consists of cutting two 1 in. wide strips of the PE pipe extending along the length of a PE pipe that contains a properly made heat fusion joint. The two strips are cut from opposing locations on the circumference of the pipe 180° apart, and each strip is 16–18 in. in length. The heat fusion joint to be tested should be approximately centered within each strip. The 1-in. wide strips may be cut from the pipe specimen using a band saw, skill saw, jig saw, or other suitable device of similar character.

The strips are then reduced to a suitable thickness for testing purposes: 0.50 in. for pipe diameters up to 12 in. nominal iron pipe size (IPS) and 0.75 in. for pipe diameters of 14 in. nominal IPS and above. The strips that contain heat fusion joints are then prepared for testing by using an electric planer to reduce the width of each strip to a uniform specimen width of 0.50 or 0.75 in. The planer also ensures that the two through-wall surfaces of the test strips are smooth in character and parallel in orientation. Tolerances for specimen preparation are ± 0.04 in. (see Fig. 8).

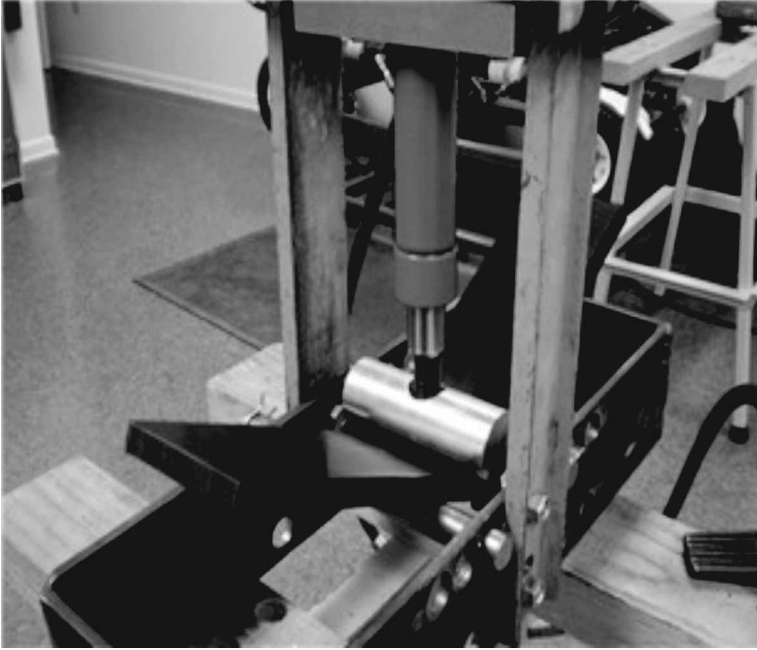


FIG. 9—GSB test apparatus with successful test result.

Test Equipment

The GSB test procedure does require some dedicated equipment of a rather simple nature. The required equipment is as follows:

- Side-bend test fixture
- Hydraulic hand pump
- Rounded ram of 1.00 and 1.50 in. diameter
- Two smooth round metal mandrels with a diameter of 0.75 in. to fit the test fixture
- Electric surface planer
- Calipers
- Measuring tape

With the exception of the GSB test fixture, all of the required equipment is generally considered “off the shelf” in nature. The GSB test fixture was developed by the task group in such a way that it could be adapted for the laboratory or the field. The test fixture holds the specimen in a specified configuration while it is fully deflected using the hydraulic hand pump. Figures 9 and 10 depict the test fixture utilized in this research program.

Test Procedure

The GSB test procedure is really quite straightforward. The GSB test fixture is securely mounted to a bench in the laboratory or other stable equipment at the



FIG. 10—GSB test apparatus with failed test result.

job site. The mandrels are then placed into the test fixture at a distance of 2.25 or 3.25 in. depending on specimen thickness: 2.25 in. for specimens that are 0.50 in. in thickness and 3.25 in. for specimens that are 0.75 in. in thickness.

The properly prepared GSB test specimen is then placed on top of the two metal mandrels in such a way that the heat fusion joint lies in the vertical plane with the centerline of the joint located directly beneath the ram that is attached to the plunger on the hydraulic cylinder. The hydraulic hand pump is then operated to lower the ram at a rate of ~ 3 in./min. As the ram is lowered, it impinges on the side-bend test specimen at the centerline of the heat fusion joint under evaluation. The GSB test specimen is then fully deflected to an arc degree of curvature in the range of 120° – 180° to induce a specific strain effect. The relationship between strain and deflection is shown as Eq (1) below

$$\text{strain (\%)} = \frac{(\text{OR} - \text{NR})}{\text{OR}} \times 100 \quad (1)$$

where:

OR = outside radius, in., = ram radius + specimen width, in., and

NR = neutral radius, in., = specimen width/[$\ln(\text{outside radius}/\text{ram radius})$].

Test failure is defined as a test specimen that breaks, tears, cracks, or splinters as a result of deflection to a specific strain rate or to a minimum arc curvature angle.

Test Report

A GSB test report for a specific heat fusion joint should include the following information:

- Identification of pipe and relevant pipe data
- OD
- Standard dimension ratio (SDR)
- Wall thickness at point from which test specimen was taken
- Pipe manufacturer and print line on pipe
- Identification of the two side-bend test specimens
- Identification of person conducting side-bend test
- Date and location of test
- Test temperature, 73°F, $\pm 2^\circ\text{F}$
- Test specimen dimensions
- Width, in inches
- Length, in inches
- Thickness (pipe wall thickness), in inches
- Ram diameter, in inches
- Rate of ram displacement, in in./min
- Calculated strain rate, in percent
- Minimum arc curvature angle, in degrees
- Pass or failure of both side-bend test specimens

Additional information can be included in the test report. However, the data shown are considered to be the minimum amount of information that should be included in the test report. Longer term, it may be feasible to standardize on a minimum arc curvature angle for each sample thickness to attain industry-established, standardized strain rates and further simplify the test method and report. It should be noted that the test temperature of 73°F designated here has been selected for purposes of standardized test conditions in an effort to assess the degree of correlation among the three test methods being discussed. As further research supports the acceptance of the GSB test method as a field-friendly test method or standard practice, it is anticipated that the actual test temperature will be variable depending on job site conditions.

Discussion

Investigation into the GSB test method started in early 2009. After initial design and equipment construction, refinements in both the testing procedures and equipment concluded with a comparative study of GSB test results to those obtained using the RB and T-I test methods.

A Commercial Study

The research program was initiated in February 2009 with the goal of determining the potential use for the GSB test methodology as an alternative means to the T-I test and the RB test for purposes of qualifying the heat fusion joining procedure on PE piping systems. The GSB test equipment used in the test program was built by the task group members. After several equipment and

TABLE 1—Comparative study test parameters.

Test Method	Test Parameters
RB test	Per ASTM F2620-06
T-I test	Per ASTM F2634-07
GSB test	Specimens per fusion weld: 2 Minimum arc degree of curvature: 120 Test temperature: 72°F±5° Test Mandrel: 0.75 in. Specimen width: 0.50 in. for pie diameters through 12 in. nominal and 0.75 in. for pipe diameters above 12 in. nominal Failure: Test specimen failure via breaking, tearing, cracking, or splintering

procedural revisions, the following test parameters were established as a basis for a comparative study of heat fusion joints made using the joining procedure detailed in ASTM F2620-06. These parameters are presented in Table 1.

To date, over 200 heat fusion test specimens taken from pipe in outside diameters from 4 in. nominal IPS to 65 in. nominal IPS and with SDRs ranging from DR 7 to DR 21 and wall thicknesses from 0.510 to 3.547 in. Testing has been performed on pipe submitted by four commercial PE pipe producers and PE pipe samples of unknown origin owing to their submittal from various customers. A portion of the testing has been conducted on fusion joints made from pipe specimens from the same producer, and another portion has been conducted on compatibility fusion joints made out of pipe specimens received from different pipe producers.

Commercial Test Data Obtained

This research was prompted by inquiries related to qualification of fusion joining procedures at job sites prior to or during the process of joining PE pipe commercially. As such, the overall test program to date would not be considered a structured test program. Rather, it should be considered a preliminary investigation into the test protocol that resulted from customer inquiries of a commercial or developmental origin. The overall data set as it currently exists is presented in the Appendix. As can be seen here, the nature of the test program is highly varied, which reflects its commercial driven nature. Pipe sizes, wall thicknesses, and pipe producers are somewhat random in nature, and in many instances, portions of data are missing. Again, this is reflective of the commercial driven nature of the program to date.

Comparative Test Data Obtained

Within the overall test results database, there exist several test regimes that represent a more comparative study of GSB testing to T-I testing and RB testing. These are test regimes 2, 5, and 7. With the exception of test regime 5,

information relative to the origin of the test samples, verification of the fusion methods utilized, and quantification of the various test results are felt to be of sufficient scale to suggest that these results could form the basis for a comparative study of these fusion joining qualification methods. It should be noted that the T-I testing in test regime 5 was not complete at the time of this writing, but it has been included here for purposes of completeness and to provide the reader with an understanding of where the research is currently focused. For ease of reference, test regimes 2, 5, and 7 are excerpted from the test results database in the Appendix and replicated here as Table 2.

General Discussion

The overall test results database is quite large and, owing to its commercially driven nature, somewhat incomplete. As noted, some of the testing was done in response to customer inquiries, and another portion was performed in the process of compliance testing for specific piping projects and/or applications. Despite the unstructured nature of some of these results, there are some notable points within the database to date, particularly as it relates to test regimes 2, 5, and 7.

Test regime 2 represents a structured test comparison that was conducted on fusions of the same size pipe (12 in. SDR 9 PE3608), from two different pipe producers, and on cross fusions made between the two pipe producers. As can be seen here, all specimens passed the RB test of ASTM F2620-06. With some expected testing variance, the test results for T-I testing per ASTM F2634-07 and the GSB test results appear to correlate well. However, T-I testing and GSB testing performed on specimens taken from the same fusion joints suggest some inherent weakness in samples AA2-1, AA2-2, BB2-1, BB2-2, AB2-1, and AB2-2 that are not revealed in the traditional RB test results. These results suggest that the T-I test and the GSB test are more rigorous in nature than the RB test and probably better suited for differentiating between acceptable and unacceptable fusion procedures in heavy-walled (> 1.00 in.) PE pipe, where acceptable fusion procedures result in a structurally sound fusion joint and unacceptable procedures result in fusion joint failure or result in joints of limited durability.

Test regime 5 is incomplete at this time. However, it should be noted that this test regime was conducted on one fusion joint of very large diameter PE3608 pipe with a wall thickness that ranged from 3.18 to 3.547 in. A total of nine fusion specimens was taken from this fusion joint and has been tested using the side-bend test protocol as outlined herein. Six of these fusion specimens pass the GSB test and three failed. Correlation testing with ASTM F2634-07 is under way at this time and should provide even more insight into the utility of the side-bend test protocol as it applies to analysis of fusion joints made within very heavy-walled pipe.

Test regime 7 is another structured comparison that reveals a very clear correlation among the test methods for qualifying heat fusion joints made using 8 in. SDR 13.5 PE3608 from one pipe producer, with all specimens passing all three test protocol. It should be noted that this particular set of tests was conducted in such a way as to emulate deviation from ASTM F2620-06 heat fusion

TABLE 2—Correlation data excerpted from the Appendix.

Test Date	Pipe Producer	Fusion Number	Specimen Number	T-I, ASTM F2634-07	RB, ASTM F2620-06	GSB
Test regime 2						
2-13-2009	AA	1	1	Pass	Pass	Pass
2-13-2009	AA	1	2	Pass	Pass	Pass
2-13-2009	AA	2	1	Pass	Pass	Fail
2-13-2009	AA	2	2	Fail	Pass	Fail
2-13-2009	BB	1	1	Pass	Pass	Pass
2-13-2009	BB	1	2	Pass	Pass	Pass
2-13-2009	BB	2	1	Fail	Pass	Pass
2-13-2009	BB	2	2	Fail	Pass	Fail
2-13-2009	AB	1	1	Pass	Pass	Pass
2-13-2009	AB	1	2	Pass	Pass	Pass
2-13-2009	AB	2	1	Fail	Pass	Fail
2-13-2009	AB	2	2	Fail	Pass	Fail
Test regime 5						
5-8-2009	AA	1	1	TBD	TBD	Pass
5-8-2009	AA	1	2	TBD	TBD	Pass
5-8-2009	AA	1	3	TBD	TBD	Fail
5-8-2009	AA	1	4	TBD	TBD	Pass
5-8-2009	AA	1	5	TBD	TBD	Fail
5-8-2009	AA	1	6	TBD	TBD	Pass
5-8-2009	AA	1	7	TBD	TBD	Pass
5-8-2009	AA	1	8	TBD	TBD	Fail
5-8-2009	AA	1	9	TBD	TBD	Pass
Test regime 7						
10-21-2009	DD	1	1	Pass	Pass	Pass
10-21-2009	DD	1	2	Pass	Pass	Pass

TABLE 2—(Continued).

Test Date	Pipe Producer	Fusion Number	Specimen Number	T-I, ASTM F2634-07	RB, ASTM F2620-06	GSB
10-21-2009	DD	2	1	Pass	Pass	Pass
10-21-2009	DD	2	2	Pass	Pass	Pass
10-21-2009	DD	3	1	Pass	Pass	Pass
10-21-2009	DD	3	2	Pass	Pass	Pass
10-21-2009	DD	4	1	Pass	Pass	Pass
10-21-2009	DD	4	2	Pass	Pass	Pass
10-21-2009	DD	5	1	Pass	Pass	Pass
10-21-2009	DD	5	2	Pass	Pass	Pass
10-21-2009	DD	6	1	Pass	Pass	Pass
10-21-2009	DD	6	2	Pass	Pass	Pass

Note: TBD means to be determined.

TABLE 3—*Test regime 7 fusion sample preparation.*

Fusion Number	Fusion Parameter Deviation
1	Control—ASTM F2620-06 fusion method
2	ASTM F2620-06 method with pressure during heat soak
3	ASTM F2620-06 method with pressure during heat soak
4	ASTM F2620-06 method with 36 s dwell time
5	ASTM F2620-06 method with 46 s dwell time
6	ASTM F2620-06 method with 45 s dwell time

procedure. All of the fusion joint specimens of test regime 7 passed all three test protocol including the more discerning ASTM F2634-07 test method. The fusion sample preparation used in test regime 7 is summarized in Table 3.

Conclusions

From this discussion, we have seen that a significant number of fusion joint evaluations have been performed by the task group. Over 200 fusion specimens have been prepared and evaluated using three different test methods: The T-I test of ASTM F2634-07, the RB test of ASTM F2620-06, and the GSB test as summarized in this paper. Based on this research, the following conclusions may be drawn.

- The data obtained to date in this program suggest a potential correlation between the GSB and T-I heat fusion joint qualification test methods.
- While it is premature at this point to conclude anything specific, the data obtained in test regime 2 suggest that the T-I test method and the GSB test method may be more discerning in their ability to identify potentially problematic heat fusion joining procedures in PE pipe.
- With additional experience and analysis of more test data, an ASTM GSB test method or standard practice may be developed as an alternative method for qualifying heat fusion joints made in PE pipe.
- The GSB test method as presented and researched here appears to hold the potential for a safe and reliable alternative means for effectively qualifying heat fusion joints in the field as compared to the traditional RB test.

It is the intent of the task group to continue a more structured research program from which very clear correlations can be drawn among the three test methods. Additionally, research into the effect of wall thickness, pipe diameter, and fusion joining parameters on GSB test results will be expanded to provide a broadened understanding of the test method.

While the testing program is not complete at this point, it has been determined from this research that the GSB test protocol may be a safe, cost-effective test method that is adaptable to the job site for purposes of fusion joint qualification as an alternative to RB or T-L testing. Further, it is expected

TABLE A1—Appendix: Guided Side-Bend Test Comparison Data.

Date or Report Number	Pipe Producer	Pipe OD and DR	Wall Thickness	Fusion Number	Specimen Number	Fusion Procedure	T-I Test Result	RB Test Result	GSB Test Result	FM Hydro
Test regime 1										
2/7/09	AA	12 in. DR 9	1.512	1	1	D2620	Pass			
2/7/09	AA	12 in. DR 9	1.512	1	2	D2620	Pass			
2/7/09	AA	12 in. DR 9	1.512	1	3	D2620	Pass			
2/7/09	AA	12 in. DR 9	1.512	1	4	D2620	Fail			
2/7/09	AA	12 in. DR 9	1.512	1	5	D2620	Pass			
2/7/09	AA	12 in. DR 9	1.512	1	6	D2620	Fail			
2/7/09	AA	12 in. DR 9	1.512	1	7	D2620	Pass			
2/7/09	AA	12 in. DR 9	1.512	1	8	D2620	Pass			
2/7/09	BB	12 in. DR 9	1.441	2	1	D2620	Pass			
2/7/09	BB	12 in. DR 9	1.441	2	2	D2620	Pass			
2/7/09	BB	12 in. DR 9	1.441	2	3	D2620	Fail			
2/7/09	BB	12 in. DR 9	1.441	2	4	D2620	Fail			
2/7/09	BB	12 in. DR 9	1.441	2	5	D2620	Pass			
2/7/09	BB	12 in. DR 9	1.441	2	6	D2620	Fail			
2/7/09	BB	12 in. DR 9	1.441	2	7	D2620	Pass			
2/7/09	BB	12 in. DR 9	1.441	2	8	D2620	Pass			
2/7/09	AB	12 in. DR 9	1.512–1.441	3	1	D2620	Pass			
2/7/09	AB	12 in. DR 9	1.512–1.441	3	2	D2620	Pass			
2/7/09	AB	12 in. DR 9	1.512–1.441	3	3	D2620	Fail			
2/7/09	AB	12 in. DR 9	1.512–1.441	3	4	D2620	Fail			
2/7/09	AB	12 in. DR 9	1.512–1.441	3	5	D2620	Fail			
2/7/09	AB	12 in. DR 9	1.512–1.441	3	6	D2620	Pass			

TABLE A1—(Continued).

Date or Report Number	Pipe Producer	Pipe OD and DR	Wall Thickness	Fusion Number	Specimen Number	Fusion Procedure	T-I Test Result	RB Test Result	GSB Test Result	FM Hydro
2/7/09	AB	12 in. DR 9	1.512–1.441	3	7	D2620	Fail			
2/7/09	AB	12 in. DR 9	1.512–1.441	3	8	D2620	Fail			
Test regime 2										
2/13/09 107	AB	12 in. DR 9	1.512–1.441	X		D2620		Pass		Pass
2/13/09 107	AB	12 in. DR 9	1.512–1.441	X		D2620		Pass		Pass
2/13/09 107	AA	12 in. DR 9	1.512	Y		D2620		Pass		Pass
2/13/09 107	AA	12 in. DR 9	1.512	Y		D2620		Pass		Pass
2/13/09 107	BB	12 in. DR 9	1.441	Z		D2620		Pass		Pass
2/13/09 107	BB	12 in. DR 9	1.441	Z		D2620		Pass		Pass
2/13/09	AA	12 in. DR 9	1.512	1	1	D2620	Pass	Pass	Pass	
2/13/09	AA	12 in. DR 9	1.512	1	2	D2620	Pass	Pass	Pass	
2/13/09	AA	12 in. DR 9	1.512	2	1	D2620	Pass	Pass	Fail	
2/13/09	AA	12 in. DR 9	1.512	2	2	D2620	Fail	Pass	Fail	
2/13/09	BB	12 in. DR 9	1.441	1	1	D2620	Pass	Pass	Pass	
2/13/09	BB	12 in. DR 9	1.441	1	2	D2620	Pass	Pass	Pass	
2/13/09	BB	12 in. DR 9	1.441	2	1	D2620	Fail	Pass	Pass	

TABLE A1—(Continued).

Date or Report Number	Pipe Producer	Pipe OD and DR	Wall Thickness	Fusion Number	Specimen Number	Fusion Procedure	T-I Test Result	RB Test Result	GSB Test Result	FM Hydro
2/13/09	BB	12 in. DR 9	1.441	2	2	D2620	Fail	Pass	Fail	
2/13/09	AB	12 in. DR 9	1.512–1.441	1	1	D2620	Pass	Pass	Pass	
2/13/09	AB	12 in. DR 9	1.512–1.441	1	2	D2620	Pass	Pass	Pass	
2/13/09	AB	12 in. DR 9	1.512–1.441	2	1	D2620	Fail	Pass	Fail	
2/13/09	AB	12 in. DR 9	1.512–1.441	2	2	D2620	Fail	Pass	Fail	
Test regime 3										
3/26/09	NA	6 in. DR 13.5		1	1	Customer	Pass	Pass	Pass	
3/26/09	NA	6 in. DR 13.5		1	2	Customer	Pass	Pass	Pass	
3/26/09	NA	6 in. DR 13.5		2	3	Customer	Pass		Fail	
3/26/09	NA	6 in. DR 13.5		2	4	Customer	Fail		Fail	
3/26/09	NA	6 in. DR 13.5		3	5	D2620			Fail	
3/26/09	NA	6 in. DR 13.5		3	6	D2620	Pass	Pass	Pass	
3/26/09	NA	6 in. DR 13.5		4	7	D2620	Fail	Pass	Pass	
3/26/09	NA	6 in. DR 13.5		4	8	D2620	Fail		Pass	

TABLE A1—(Continued).

Date or Report Number	Pipe Producer	Pipe OD and DR	Wall Thickness	Fusion Number	Specimen Number	Fusion Procedure	T-I Test Result	RB Test Result	GSB Test Result	FM Hydro
3/36/09	NA	6 in. DR 13.5		5	9	D2620	Fail		Pass	Pass
3/26/09	NA	6 in. DR 13.5		5	10	D2620			Fail	Fail
Test regime 4										
4/25/09	NA	36 in. DR 17		6	1	D2620	Pass		Pass	Pass
4/25/09	NA	36 in. DR 17		6	2	D2620	Pass		Pass	Pass
4/25/09	NA	36 in. DR 17		6	3	D2620	Pass		Pass	Pass
4/25/09	NA	36 in. DR 17		6	4	D2620	Pass		Pass	Pass
4/25/09	NA	36 in. DR 17		7	1	D2620	Pass		Pass	Pass
4/25/09	NA	36 in. DR 17		7	2	D2620	Pass		Pass	Pass
4/25/09	NA	36 in. DR 17		7	3	D2620	Pass		Pass	Pass
4/25/09	NA	20 in. DR 9		8	1	D2620	Pass		Pass	Pass
4/25/09	NA	20 in. DR 9		8	2	D2620	Pass		Pass	Pass
4/25/09	NA	20 in. DR 9		8	3	D2620	Pass		Pass	Pass
4/25/09	NA	20 in. DR 9		8	4	D2620	Pass		Pass	Pass

TABLE A1—(Continued).

Date or Report Number	Pipe Producer	Pipe OD and DR	Wall Thickness	Fusion Number	Specimen Number	Fusion Procedure	T-J Test Result	RB Test Result	GSB Test Result	FM Hydro
Test regime 5										
5/8/09 114	AA	65 in. DR 21	3.547	1	1	D2620			Pass	
5/8/09 114	AA	65 in. DR 21	3.547	1	2	D2620			Pass	
5/8/09 114	AA	65 in. DR 21	3.547	1	3	D2620			Fail	
5/8/09 114	AA	65 in. DR 21	3.547	1	4	D2620			Pass	
5/8/09 114	AA	65 in. DR 21	3.547	1	5	D2620			Fail	
5/8/09	AA	65 in. DR 21	3.547	1	6	D2620			Pass	
5/8/09	AA	65 in. DR 21	3.547	1	7	D2620			Pass	
5/8/09	AA	65 in. DR 21	3.547	1	8	D2620			Fail	
5/8/09	AA	65 in. DR 21	3.547	1	9	D2620			Pass	
Test regime 6										
8/17/09 115	CC	18 in. DR 7		1	1	D2620		Pass	Pass	
8/17/09 115	CC	18 in. DR 7		1	2	D2620		Pass	Pass	

TABLE A1—(Continued).

Date or Report Number	Pipe Producer	Pipe OD and DR	Wall Thickness	Fusion Number	Specimen Number	Fusion Procedure	T-I Test Result	RB Test Result	GSB Test Result	FM Hydro
8/17/09 115	CC	18 in. DR 7		2	1	D2620		Pass	Pass	
8/17/09 116	CC	18 in. DR 7		2	2	D2620		Pass	Pass	
8/17/09	CC	4 in. DR 9		3	1	D2620		Pass	Pass	
8/17/09		4 in. DR 9		3	2	D2620		Pass	Pass	
8/17/09	CC	6 in. DR 9		4	1	D2620		Pass	Pass	
8/17/09		6 in. DR 9		4	2	D2620		Pass	Pass	
8/17/09	CC	8 in. DR 9		5	1	D2620		Pass	Pass	
8/17/09		8 in. DR 9		5	2	D2620		Pass	Pass	
8/17/09	CC	10 in. DR 9		6	1	D2620		Pass	Pass	
8/17/09		10 in. DR 9		6	2	D2620		Pass	Pass	
8/17/09	CC	12 in. DR 9		7	1	D2620		Pass	Fail	
8/17/09		12 in. DR 9		7	2	D2620		Pass	Fail	
8/17/09	CC	14 in. DR 9		8	1	D2620		Pass	Pass	
8/17/09		14 in. DR 9		8	2	D2620		Pass	Pass	
8/17/09	CC	16 in. DR 9		9	1	D2620		Pass	Pass	
8/17/09		16 in. DR 9		9	2	D2620		Pass	Pass	
8/17/09	CC	18 in. DR 9	2.874			D2620		Pass	Fail	
8/17/09		18 in. DR 9	2.874			D2620		Pass	Fail	

TABLE A1—(Continued).

Date or Report Number	Pipe Producer	Pipe OD and DR	Wall Thickness	Fusion Number	Specimen Number	Fusion Procedure	T-J Test Result	RB Test Result	GSB Test Result	FM Hydro
Test regime 7										
10- 21weld 1	DD	8 in. DR 13.5	0.655	1	1	D2620	Pass	Pass	Pass	
10- 21weld 1	DD	8 in. DR 13.5	0.655	1	2	D2620	Pass	Pass	Pass	
10/ 21weld 5 2a	DD	8 in. DR 13.5	0.655	2	1	Ht w pressure	Pass	Pass	Pass	
10/ 21weld 5 2a	DD	8 in. DR 13.5	0.655	2	2	Ht w pressure	Pass	Pass	Pass	
10/ 21weld 7 2c	DD	8 in. DR 13.5	0.655	3	1	Ht w pressure	Pass	Pass	Pass	
10/ 21weld 7 2c	DD	8 in. DR 13.5	0.655	3	2	Ht w pressure	Pass	Pass	Pass	
10/ 21weld 21 7a	DD	8 in. DR 13.5	0.655	4	1	36 s dwell	Pass	Pass	Pass	
10/ 21weld 21 7a	DD	8 in. DR 13.5	0.655	4	2	36 s dwell	Pass	Pass	Pass	

TABLE A1—(Continued).

Date or Report Number	Pipe Producer	Pipe OD and DR	Wall Thickness	Fusion Number	Specimen Number	Fusion Procedure	T-I Test Result	RB Test Result	GSB Test Result	FM Hydro
10/ 21weld24 8a	DD	8 in. DR 13.5	0.655	5	1	46 s dwell	Pass	Pass	Pass	
10/ 21weld 24 8a	DD	8 in. DR 13.5	0.655	5	2	46 s dwell	Pass	Pass	Pass	
10/ 21weld25 8b	DD	8 in. DR 13.5	0.655	6	1	46 s dwell	Pass	Pass	Pass	
10/ 21weld 25 8b	DD	8 in. DR 13.5	0.655	6	2	46 s dwell	Pass	Pass	Pass	

that this extended research may form the basis for development of an ASTM GSB test method or standard practice.

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Tensile Testing of a Push-On Restrained Joint PVC Pipe

ABSTRACT: Pipes installed using trenchless methods such as horizontal directional drilling (HDD) are subjected to tensile forces during installation. Polyvinyl chloride (PVC) pipe employing the Bulldog Restraint System (BRS) is being used for installation via HDD. Tensile tests were conducted for assemblies of PVC pipe 4 in. through 8 in. in diameter using BRS. The test specimens were subjected to constant displacement; load and strain were measured. The observations from and results of the tests are reported in this paper.

KEYWORDS: PVC pipe, horizontal directional drilling (HDD), internally restrained joint

Introduction

Restraint mechanisms internal to the bell of a pipe or the fitting were first developed for iron pipe in Germany by Duker over 20 years ago. U.S. Pipe (an iron pipe manufacturer) introduced a restrained gasketed system to the U.S. pipe market in the mid-1980s. Recently, a similar system has been developed for polyvinyl chloride (PVC) piping that goes by the brand name of Bulldog. This joint restraint system is integral to the bell of a PVC pressure pipe or fabricated fitting and is incorporated into the product during its manufacture.

The current version of the Bulldog Restraint System (BRS) is designed for integration into AWWA C900 PVC pipe and fittings in diameters of 4 in. through 12 in. The mechanism consists of a metal casing that sits adjacent to the Rieber

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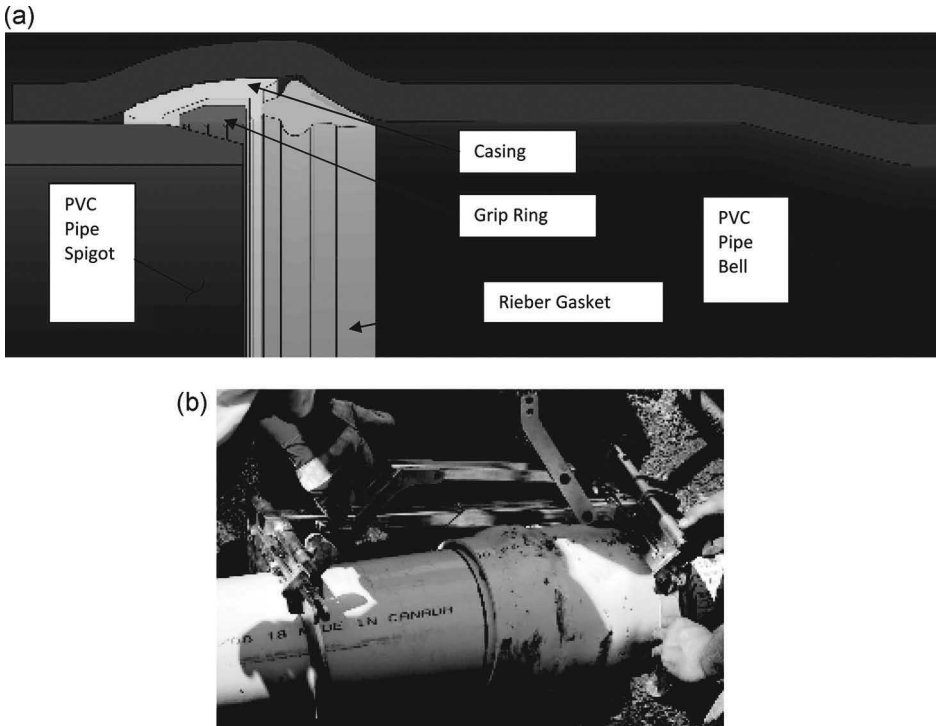


FIG. 1—(a) Integral PVC joint restraint components. (b) Assembled push-on restrained joint.

gasket in the bell, and the casing is molded into the raceway of the bell during pipe belling. A C-shaped grip ring with several rows of uni-directional serrations is manually inserted into the casing at the manufacturing facility. Both the casing and the grip ring are made of ductile iron that has been coated using an electro-coating process that achieves a uniform thickness and provides superior corrosion resistance. When the pipe arrives at a jobsite, the bell already contains the casing with the grip ring inserted in it, and no additional hardware is needed in order to provide a restrained joint. Figure 1(a) is a cross-sectional view of the components in the joint. In the field, the joint is assembled like a regular push-on joint: The spigot is pushed into the bell up to the insertion mark. Figure 1(b) shows an assembled joint and a manual pipe-puller that makes assembly easier.

First Round of Tensile Testing

One of the applications for which this product is used is horizontal directional drilling (HDD). For the HDD contractor, the tensile strength of the restrained joint is an important piece of information. Knowledge of this value allows the contractor to plan pulls that are within the product's capabilities and which have an adequate margin of safety.

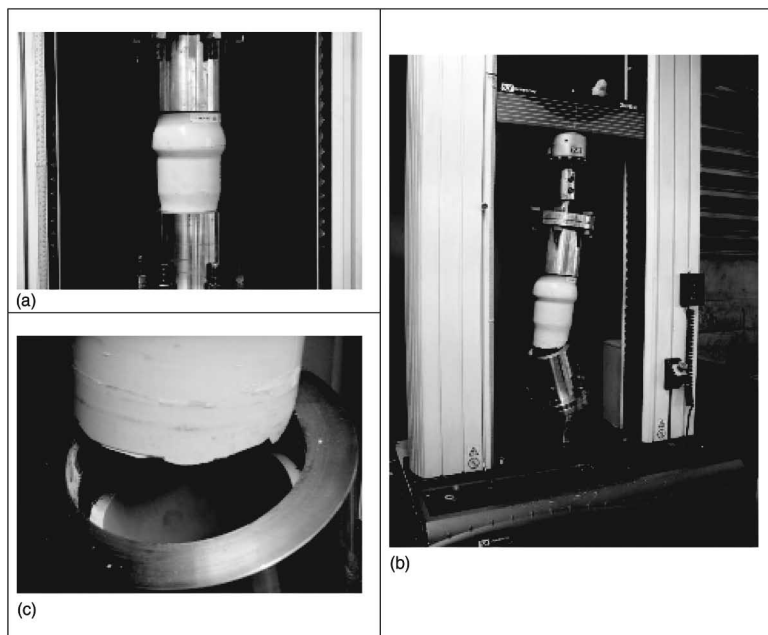


FIG. 2—The first end grip design proved weaker than the BRS. (a) Test setup. (b) The end grip sheared the assembly before the restrained joint failed. (c) The failure from the end grip was a circumferential shear failure.

Acquiring tensile strength data proved a bit challenging. In the first round of testing conducted by S&B Technical Products, the Research and Development Department found that the internal restraint device's gripping mechanism was more effective at transferring the tensile loads than the grips at the end of the specimen were. The end grip sheared the PVC prior to attaining the restraint device's tensile strength (see Fig. 2). Remember the difficulty that the designer faced: He had to design an end fixture for an unknown tensile strength load.

Successful Connection Methods

Meanwhile, licensees of the technology were experimenting with their own connection techniques. One licensee doubled up a wedge-activated external restraint harness. The two restraints were connected to each other and to a steel plate by threaded rods (see Fig. 3).

The licensee's end-grip solution allowed the researcher to collect data on the tensile strength of the pipe in all sizes. The next phase of the investigation was to collect additional engineering data along with the tensile strength. Also, an HDD pulling-head design would be proven while the data were being collected.

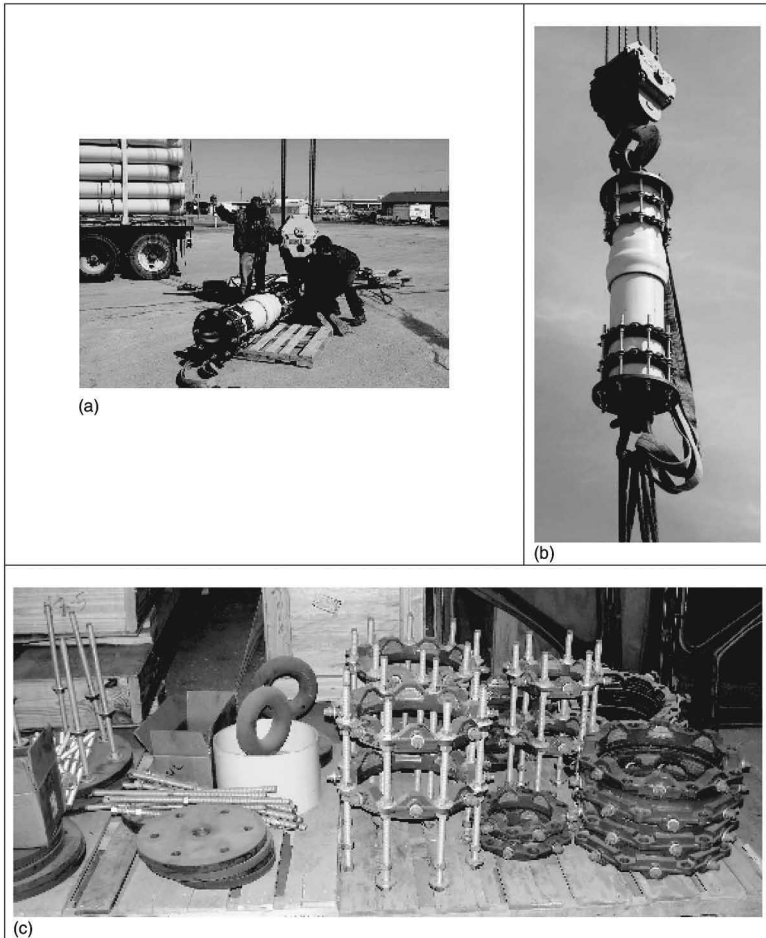


FIG. 3—End grip hardware. (a) The connection method devised was also used for a marketing photo, which demonstrated the product's strength by lifting a load of pipe. (b) Close-up of the doubled-up restraint devices that were rodde to a steel plate. (c) End-grip hardware used by a licensee for acquiring pull-apart strength data for all sizes produced.

University of Texas at Arlington Tensile Testing

S&B Technical Products contracted with the University of Texas of Arlington (UTA) for the next round of testing. At UTA, the testing was coordinated by the Center for Underground Infrastructure Research and Education and conducted at the Civil Engineering Laboratory Building (CELB). Most of the information on the testing is drawn from the UTA test report [1].

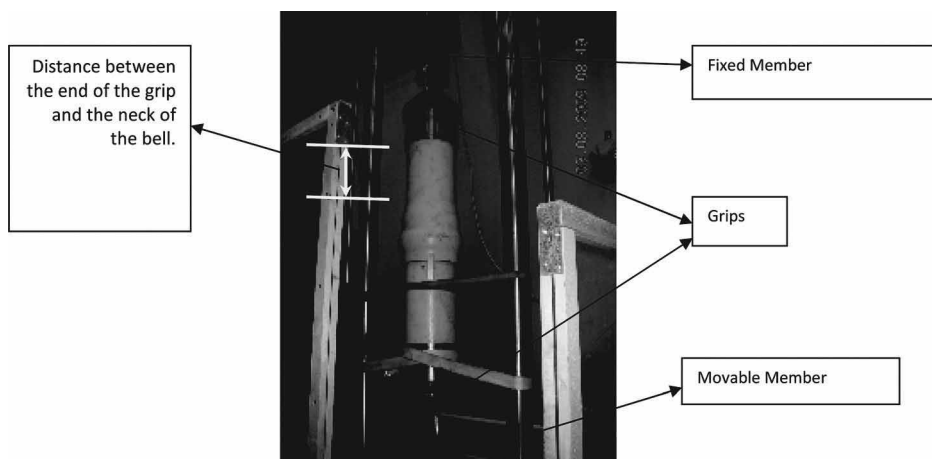


FIG. 4—Test apparatus used for determining the pull-apart strength of an 8 in. product.

Testing Apparatus

The testing apparatus for testing the product at diameters of 4 in. through 8 in. is shown in Fig. 4. Testing apparatus details are as follows:

Testing Machine—An MTS 810 material test system with a capacity of 100 kips was used for the test. It comprises (1) a fixed member, (2) a movable member, and (3) grips. The fixed member was stationary and at the top end of the machine; it had one of the grips affixed to it. The movable member was at the bottom of the machine and had the other grip affixed to it.

Drive Mechanism—A uniform, controlled velocity was applied to the movable member with respect to the fixed member. The velocity used during the tests was 0.2 in. of displacement per minute.

Load Indicators—Material test system (MTS) load cells with a capacity of 110 kips were used to determine the total tensile loads carried by the test specimen.

Strain Gages—Two rectangular strain gage rosettes were attached onto the specimen in order to measure the strain and strain directions on the specimen. One assembly of strain gage rosettes was placed on the bell side of the pipe assembly, and the other was on the spigot side of the pipe assembly, as shown in Fig. 5.

Grips—Grips held the test specimen between the fixed and the movable member of the testing machine. In this test, fixed grips that made uniform serrations in order to prevent the slippage of the specimen were used to hold the



FIG. 5—Strain gage rosettes attached to specimens collected strain data.

specimen rigidly between the fixed and the movable member. These grips were purchased from DCD Design and Manufacturing of Richmond, British Columbia, Canada. The grips are commercially available pulling heads used for installing thermoplastic pipe via HDD. Figure 6 shows a photo of the grip prior to installation.



FIG. 6—End grip used for a 4 in. specimen.

Specimens

Pipe specimens were supplied by two different PVC pipe manufacturers. The specimens were sent to the CELB unassembled and with equal numbers of bell and spigot ends. The pipe specimens were cut to appropriate sizes so that the pipe assembly length would be as desired for the test. Specimens with larger diameters had longer assembly lengths in order to keep a minimum distance between the end of the grip and the neck of the bell (see Fig. 4). The lengths of the pipe assemblies tested are given in Table 1. Rectangular strain gage rosettes were carefully attached to the specimen (Fig. 5). Bell and spigot sections were assembled using an MTS compression machine. Grip heads were inserted into both ends of the pipe assembly and tightened as recommended by the supplier of the grips. The end of the external steel sleeve was marked on the pipe in order

TABLE 1—Details of specimens used for the test.

Specimen Number	Nominal Pipe Diameter, in.	Length, in.	Minimum Wall Thickness per C900, in.	Dimension Ratio
1	4	36	0.267	18
2	4	36	0.267	18
3	6	46	0.383	18
4	6	46	0.383	18
5	8	46	0.503	18
6	8	46	0.503	18

TABLE 2—Summary of results.

Specimen Number	Nominal Pipe Diameter, in.	Failure Location	Load at Failure, lb	Displacement at Failure, in.	Longitudinal Strain at Failure, μ -in/in		
					Bell End	Spigot End	
1	4	Bell	19 479	1.67	10 259	14 059	
2	4	Spigot	21 237	2.48	14 044	14 044	
3	6	Bell and spigot	38 163	2.02	Not Instr.	Not Instr.	
4	6		35 722	1.69	Not Instr.	Not Instr.	
5	8		54 249	1.37	4 315	10 001	
6	8		54 181	1.24	4 335	10 175	
Principal Strain (Bell End), μ -in/in					Principal Strain (Spigot End), μ -in/in		
Specimen Number	Nominal Pipe Diameter, in.	Max.	Min.	Direction	Max.	Min.	Direction
1	4	10 333	-4103	Circ.	14 065	-4690	Circ.
2	4	14 029	-5224	Long.	3023	-1560	Long.
3	6	Not Instr.	Not Instr.	Not Instr.	Not Instr.	Not Instr.	Not Instr.
4	6	Not Instr.	Not Instr.	Not Instr.	Not Instr.	Not Instr.	Not Instr.
5	8	4336	-5017	Circ.	10 012	-4106	Circ.
6	8	4359	-5631	Circ	10 181	-3549	Long.

Note: Not Instr = Not instrumented with strain gages; Max = Maximum; Min = Minimum; Circ = Circumferential; Long = Longitudinal.

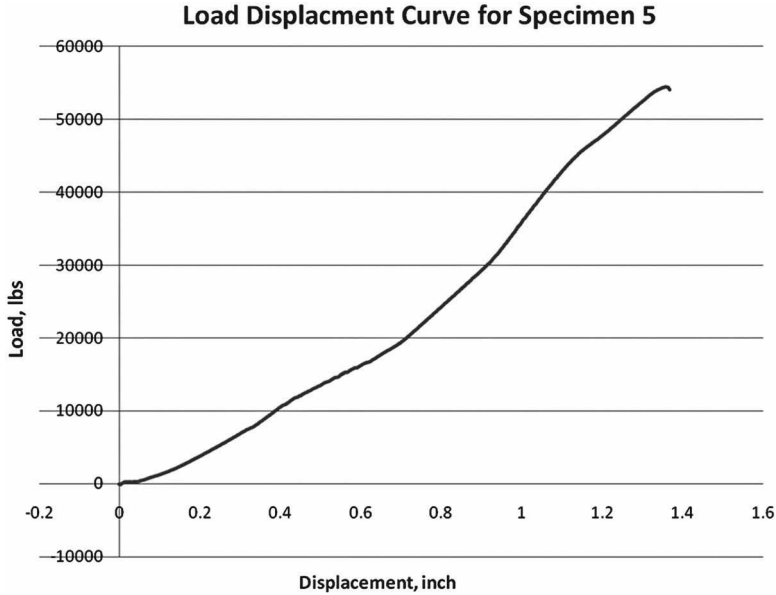


FIG. 7—Load-displacement curve for specimen 5.

to monitor any slippage. The specimens were loaded into the MTS machine as shown in Fig. 4.

Data and Analysis

For each specimen, the following graphs were prepared:

- Load curve: Time in seconds is the independent variable, and load in pounds is the dependent variable.
- Displacement curve: Time in seconds is the independent variable, and displacement in pounds is the dependent variable.
- Load-displacement curve: Displacement in inches is the independent variable, and load in pounds is the dependent variable.
- Principal strain curve, bell end: One curve plotted on this graph is the maximum principal strain versus load. The maximum principal strain in micro-inches per inch is the independent variable, and load in pounds is the dependent variable. The second curve plotted is the minimum principal strain versus load. The minimum principal strain in micro-inches per inch is the independent variable, and load in pounds is the dependent variable.
- Principal strain curve, spigot end: The same two curves described above for the bell end are plotted for the spigot end.

The other data collected were the failure location and the displacement at failure.

A more detailed discussion of the data collected and the test results is provided in Ref. 1. A summary of the data collected is provided in Table 2.

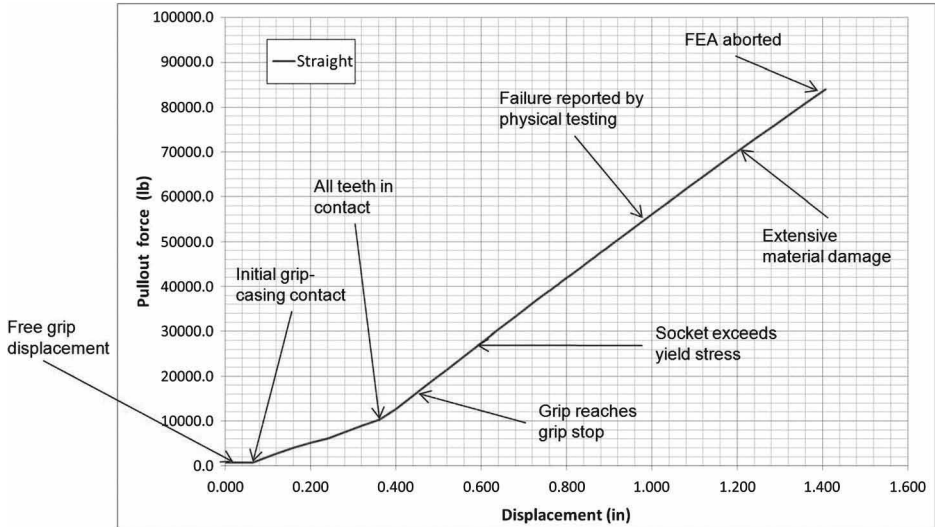


FIG. 8—Load-displacement curve from FEA for an 8 in. assembly [2].

Discussion

Shape of the Load-Displacement Curve

The load-displacement curve shows (by its slope) the consequences of the various positions of the grip ring as it engages. Figure 7 shows the actual load-displacement data from the test of an 8 in. diameter assembly. Figure 8 gives the same information generated by a finite element analysis (FEA) computer model for an 8 in. assembly. The sharp changes in slope in the FEA-produced load-displacement curve were a response to the grip ring's position as it was activated.

There is very good agreement between the physical test and the FEA analysis, particularly regarding the changes in slope that occur, first when the grip reaches its mating conical surface on the casing and again when the grip approaches the grip stop. Notice, however, that there is a considerable difference in the displacement scale. Whereas physical testing reports failure at a displacement of 1.3 in. with a 54 000 lb load, the equivalent condition in the FEA shows a 1.0 in. displacement. This difference is explained by the difference in the distance between the points where the displacement is measured. Greater separation introduces greater pipe elongation, which, added to the displacement at the restraint itself, leads to reports of greater displacement.

Refer to Fig. 9 to see the grip ring positions that correspond to various points on the load-displacement curve given in Fig. 8.

Failure Modes Observed

Figure 10 shows the stresses that the bell-and-spigot assembly is experiencing at the time of failure. The regions of highest stress are the regions in which failure occurred during the physical testing. One failure mode was the shearing of the

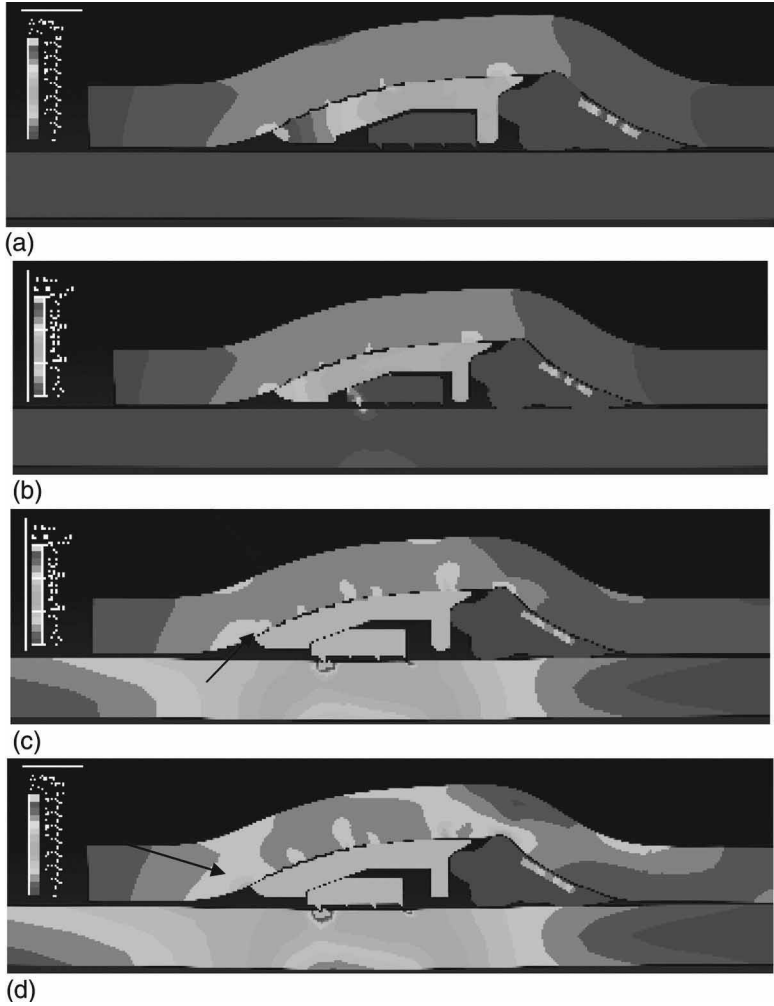


FIG. 9—Grip ring positions during activation. (a) After spigot insertion, the grip ring is away from the mating conical surface on the grip. (b) There is free movement of the grip ring until it comes in contact with the casing. (c) All grip ring teeth are in contact with the spigot. (d) The grip ring reaches the grip stop wall on the casing [2].

spigot underneath the leading tooth. The other failure mode observed was the failure of the bell above the apex of the gasket. Table 2 lists which failure modes were dominant in the physical test. Photos of specimens that underwent each type of failure mode are shown in Fig. 11.

Gripping Mechanism

The teeth of the internal pulling head (also called a grip in the Testing Apparatus section) left very shallow indentations on the inside diameter of the pipe

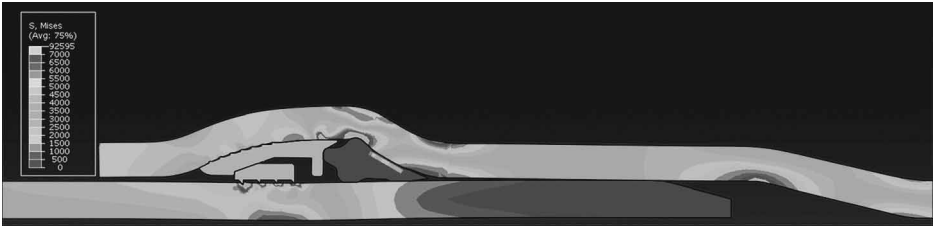


FIG. 10—FEA of an 8 in. assembly at failure load [2].

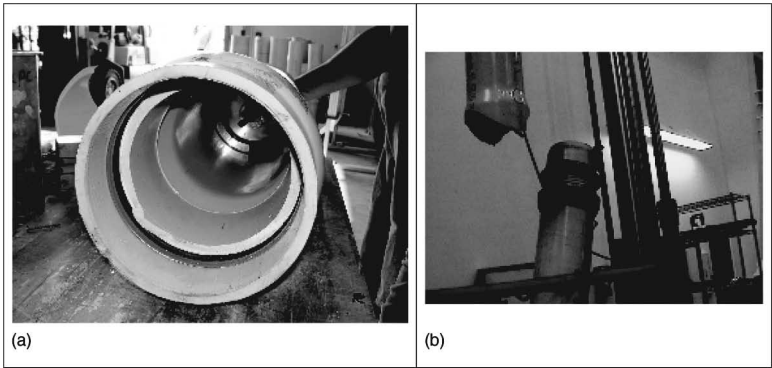


FIG. 11—Photos of the two failure modes observed. (a) The spigot was sheared and the bell remained intact. (b) The bell failed and the spigot remained intact.



FIG. 12—Indentations left on the internal diameter of the pipe.

(Fig. 12). Given the large failure loads achieved, these minor indentations are impressive. The tensile testing demonstrated that this pulling head is able to achieve the full tensile strength of the BRS product.

Conclusion

The tensile testing conducted by UTA on the PVC pipe employing the BRS has demonstrated that these products have the tensile strength needed for HDD applications. This can be verified by comparing the tensile strengths in this paper with those of other thermoplastic products being used in HDD applications. Another means of verifying this is to compare the BRS tensile strengths to loads predicted using ASTM F1962. The testing also has proven a particular style of HDD pulling head. For trenchless applications, it is also important to know about the behavior of a joint under a combination of tensile and bending loads. This testing will be the subject of another report.

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Building Knowledge from Failure Analysis of Plastic Pipe and Other Hydraulic Structures

ABSTRACT: Structural plastics have long shown exceptional performance in products used for fluid transport and containment, such as pipes, tanks, and drainage structures. The development and use of plastics enhances the versatility and durability of these important infrastructure components, as well as their economy. However, like all materials, plastics have performance limitations, and serious failures can occur as the result of improper design, fabrication, or installation. Understanding failure mechanisms of plastics can provide valuable insight into those limitations and aid the designer, manufacturer, and installer in optimizing structural and functional performance. We review forensic techniques, including field investigation, material and mechanical laboratory tests, optical and scanning electron microscopy, and stress analysis. The example cases highlight mechanical, environmental and installation issues. These include failure to account for material creep and fatigue, defects in fusion bonding, incompatibility of solvent cement, strength limitation of joint elements, and improper installation techniques. The case studies suggest ways to avoid similar mistakes in the future.

KEYWORDS: structural plastic, creep, fatigue, installation, investigation

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Introduction

Structural plastics are the preferred class of material for a multitude of piping, storage, and drainage applications of gas and fluids such as natural gas, liquid fuel, and water. Structural plastics are widely used in underground, above-ground, and building piping systems. Potable water lines, hydronic water lines, sewer lines, and fire protection water lines are commonly made out of structural plastics.

Structural plastics offer many advantages over metallic materials in piping applications. Unlike metals, plastics are highly resistant to water and soil environments. Under normally corrosive conditions, their service life is not dictated by the rate of thickness loss, which is a typical concern with steel and cast iron. From a fabrication standpoint, complex mold shapes can be produced with sufficient precision to eliminate the need for machining. In many cases, structural plastic products are also easier to install because of their light weight and flexibility relative to other pipe materials.

Despite their numerous advantages, a small fraction of structural plastic products do fail in service. These failures often result from the lack of design knowledge about the behavior of plastic materials over their intended service life. Even when properly designed for the intended application, plastic materials can fail due to improper installation or misuse in service. This paper describes the typical approach to failure analysis, discusses the general types and categories of failures encountered with structural plastic products, and presents several case studies that provide lessons for avoiding similar failures in the future.

General guidelines are available to aid in conducting a failure investigation and identifying cause [1,2]. The scope of a failure analysis ranges from rudimentary to elaborate depending on the complexity and importance of the part or system under investigation. A typical failure analysis includes the following tasks:

- *Information review:* Review relevant documents related to the design, manufacturing, installation, and service use. Important information includes
 - Test data, including from material, mechanical, and full-scale trial tests;
 - Design calculations;
 - Manufacturing quality control procedures and records;
 - Governing standards and codes;
 - Project specifications and drawings;
 - Installation and field testing reports; and
 - Photographs of the installation and the failure.

Interviews of the user, the installer, the manufacturer, and the designer may yield additional insight.

- *Visual examination and measurement of the failure site:* Examine the failed component and the system in which it is operating. This examination may reveal clues on unusual loading or other service conditions that triggered the failure. Physical measurements of dimensions and loads may be useful for load analysis and calculations. In addition, markings on pipes and a review of the operational conditions can provide information about the material specifications, service temperatures, and applied pressures.

- *Sampling*: Obtain samples of the failed pieces and, if necessary, of reference pieces that have not failed. When available, control samples provide a reference for comparing properties of failure samples with expected conditions. Depending on the objective of the failure analysis, a statistical approach may be required to determine the minimum sample size needed to verify a hypothesis.
- *Microscopy*: Examine the failed component by optical microscopy or by scanning electron microscopy (SEM). Examinations of the fracture surface may require cleaning in mild solutions with ultrasonic vibrations to remove residues. Cutting and polishing, using accepted methods of metallographic sample preparation, can reveal the surface of the material or joint in cross-section.
- *Material and mechanical testing*: Conduct tests to verify that the material and finished product meet the requirements of applicable specifications and design intent as well as to identify inherent defects that relate to the cause of failure. Tests in addition to those required by product specifications include
 - Material identification using such techniques as elemental dispersion X-ray, Fourier transform infra-red (FTIR) spectroscopy, and gas chromatography;
 - Thermal analysis to determine temperature-related behavior using differential scanning calorimetry and thermogravimetric analysis;
 - Environmental exposure to determine property changes due to chemical immersion, ultraviolet radiation, or tests for environmental stress crack resistance; and
 - Creep and fatigue tests to determine response to long-term and cyclic loading.
- *Stress analysis*: Conduct stress analyses to evaluate conditions of overload, under capacity or concentration of stress that may have contributed to failure. Depending on the complexity of the system in which the part is operating, analyses may vary from simple code-based hand calculations to sophisticated, computer-based finite element modeling.

In some cases, laboratory assembly testing and/or computer simulation supplement a typical failure analysis to determine the contributing effects of design, fabrication, installation, and service use. For example, analysis of pipe joint failures often necessitates full- or partial-scale testing under service conditions, supplemented by finite element analysis of the joint to correlate failure mechanism with peak stresses and deformation at the failure initiation site. Often, literature data from reference texts, handbooks, journal articles, or other published sources will help to interpret the results from the information review, examination, and testing.

Failure Categories

Conventionally, the cause of a failure can be classified as either a “proximate” cause or a “root” cause. A proximate cause (or direct cause) is a condition that existed, or event that occurred, immediately before a failure that directly

resulted in the failure, and if removed or altered would have prevented the failure from occurring. A root cause refers to an organizational or operational problem that led to the proximate cause. Future failures are best avoided by identifying and addressing root causes. For example, the proximate cause of a pipe joint leak may be inadequate bond due to degraded solvent cement, whereas the root cause is a deficient quality control program for monitoring the shelf life of the cement.

Often, a failure can be attributed to more than one cause. For instance, a burst pipe may be found non-conforming to a specification for minimum strength, while post-failure design review also reveals a demand higher than the rated capacity of the pipe.

Failure investigations do not always determine all the contributing causes of failure, nor do they always extend to identification of root causes. Investigators can identify probable proximate causes based on available evidence, experience, and systematic elimination of other causes. For example, it may not be possible to determine the proximate cause of a rolled gasket that resulted in a pipe joint leak. However, after eliminating material and dimensional defects through measurement and testing, the evidence from physical examination and interviews with pipe installers could suggest improper pipe alignment during jointing as the probable contributing proximate cause and poor employee training as the root cause.

The results of a failure analysis can provide valuable information with respect to improving material applications, standards and specifications, as well as design and installation procedures. The case studies discussed herein identify causes of failures and lessons learned from several investigations of plastic pipe and other hydraulic structures.

For this presentation, we categorize failures under the general headings of mechanical, environmental compatibility, and installation. Although these categories are not necessarily exclusive, they provide a framework for a discussion of causation. Mechanical failures include cases where the load demand exceeds the capacity without apparent environmental compatibility or installation issues. Failures associated with environmental compatibility include those due to chemical degradation in service or improper material selection for the intended or actual environment. Installation-related failures comprise those resulting from deficient installation specifications or faulty construction.

Case Studies

Below we discuss the findings of example failure investigations of plastic pipe and other hydraulic structures.

Case Study 1—Deformation and Collapse of Thermoplastic Storm Retention Units

Shortly after installation of a subsurface thermoplastic storm retention system, portions of the system were found to have partially collapsed beneath a parking lot. It was believed that excessive construction loads may have contributed to the failure. Excavation of distressed areas revealed large lateral and vertical deflections of the structure (Fig. 1). Review of manufacturer's documents



FIG. 1—*Deformed and collapsed units.*

indicated field tests for short-term vertical loads, but no direct evaluation of the effects of sustained lateral loading on the installed system.

Forensic test pits of non-collapsed sections of the structure revealed inward deformation of perimeter units (Fig. 2) and of buckled/broken struts (the horizontal links between adjacent cores) (Fig. 3).

Tests for basic material and mechanical properties did not show any obvious material defects; however, field samples were more brittle in tension than new samples obtained from the manufacturer (Fig. 4).

Buckling strength calculations for the struts indicated possible under capacity particularly when accounting for creep effects due to lateral loads from soil and surface vehicles. There was no evidence from design documents that the effects of strut buckling due to creep loads were considered. In buried structures, both the soil overburden and parked vehicles can impose sustained loading on the structure. Because buckling strength is highly dependent on the rotational stiffness of strut ends, tests were conducted to confirm actual buckling strength (Fig. 5).

Short-term load tests showed that struts behaved more like pinned-end rather than fixed-end elements. Creep tests confirmed that the buckling capacity drops significantly in response to long-term sustained loading. In both the short-term and creep tests, brittle fracture occurred at the strut ends as the struts buckled (Fig. 6).

Extrapolation of creep data shows about a fivefold decrease in buckling strength at 50 years (Fig. 7). This compares well to current practice of using creep modulus values in standard buckling equations [3].

Implications from this investigation include:

- Design of the retention structures must account for lateral loads—including soil and vehicle loads;



FIG. 2—*Lateral deformation at perimeter units.*

- Load testing of manufactured pieces is needed to corroborate design analysis; and
- The design must account for long-term creep behavior and propensity for brittle fracture.

In the absence of standards for design and testing, the user has limited guidance beyond the claims of the manufacturer to judge the structural viability of these types of buried structures. ASTM F2418 and F2787 are examples of standards that have recently been developed to address these needs for corrugated, arch-type, and storm retention chambers [4,5].

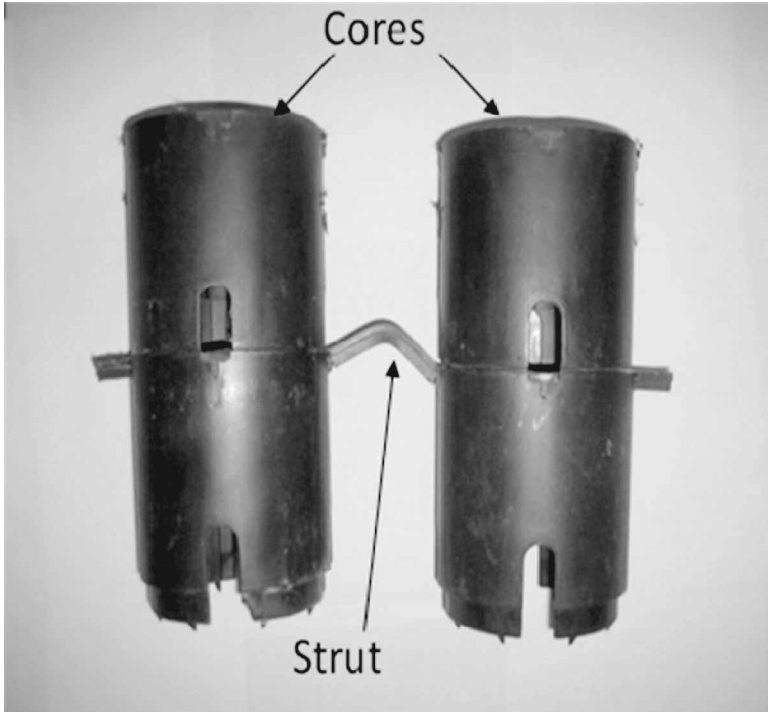


FIG. 3—Buckled strut between two columns of cores.

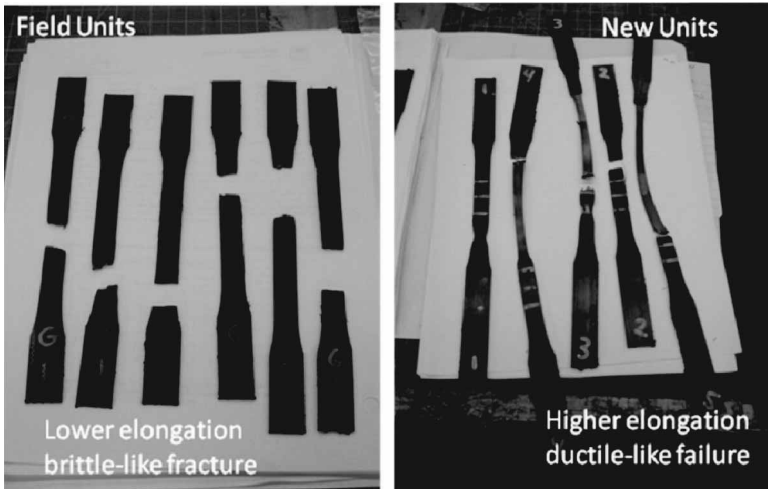


FIG. 4—Tensile coupons from field and new units showing brittle behavior of field units.

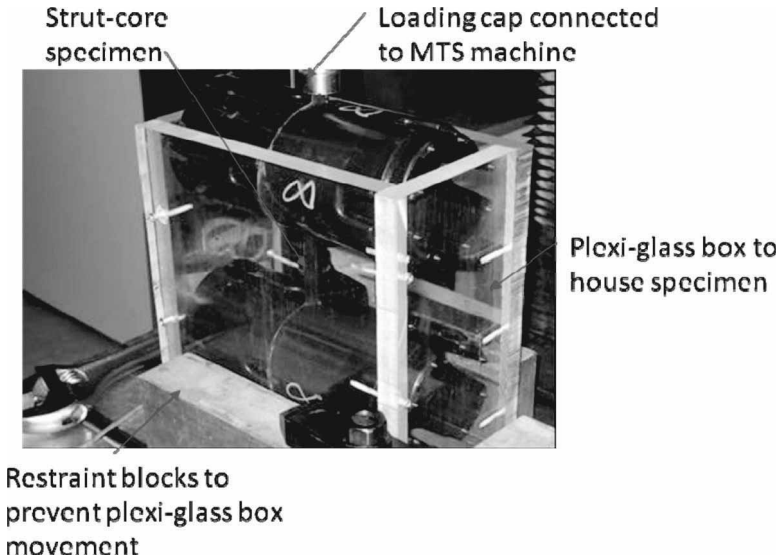


FIG. 5—Test setup for buckling tests of strut between two cores.

Case Study 2—Longitudinal Cracking of CPVC Sprinkler Pipes

Shortly after installation, pipe sections of a chlorinated poly-vinyl chloride (CPVC) sprinkler system began to exhibit longitudinal cracks, away from solvent-cemented joints. Figures 8 and 9 show the main crack, secondary cracks, and the interior surface with localized darkening. The darkening corresponds to an oily film that tended to penetrate into the cracks.

We obtained a section of pipe that was supplied to the project, but never installed. We tested the material for creep resistance using a split-disk configuration (Fig. 10). The pipe showed no signs of insufficient creep resistance. The material also had sufficient short-term strength.



FIG. 6—Brittle fracture at ends of buckled strut.

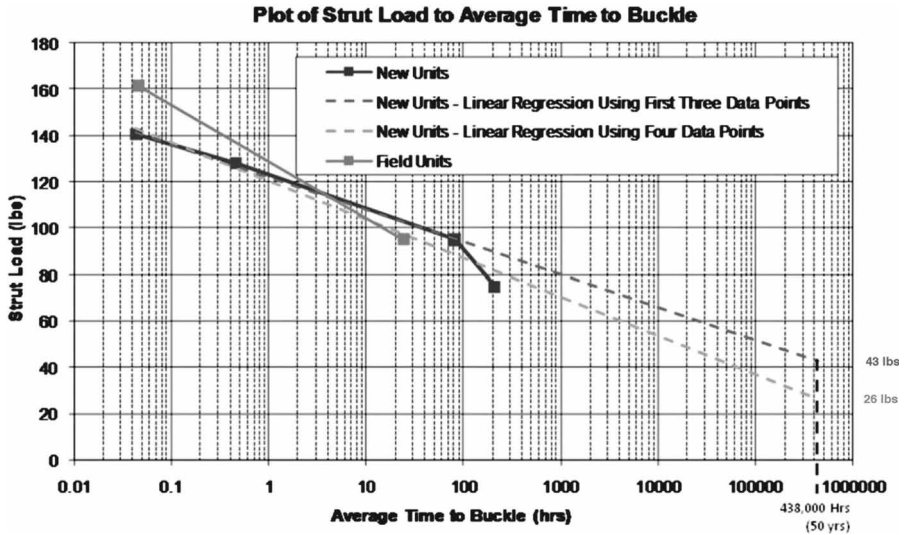


FIG. 7—Creep test data extrapolated to obtain approximate 50 year strength (Note: Longer term test data are needed to confirm 50 year strength).

We hinged open the main crack of a field sample to examine the appearance. Figure 11 shows both sides of the cracked surface where the outside surfaces of the pipe segments are touching each other. The bright regions correspond to the fracture surface created in the laboratory. The darker regions indicate slow crack growth from the pipe interior. For this section of pipe, the area of failure by slow crack growth is smaller than the area corresponding to the fracture cause by hinging the sample.

Figure 12 shows one-half of the pipe in the same configuration as in Fig. 11. This photo was taken closer to the crack initiation site. The areas of bright material in this sample indicate that only a small amount of material was separated as a result of the laboratory testing. The remainder of the material (the darker regions) indicates in-service slow crack growth that progressed through the thickness of the pipe wall in some locations. The vertical lines at the inner pipe

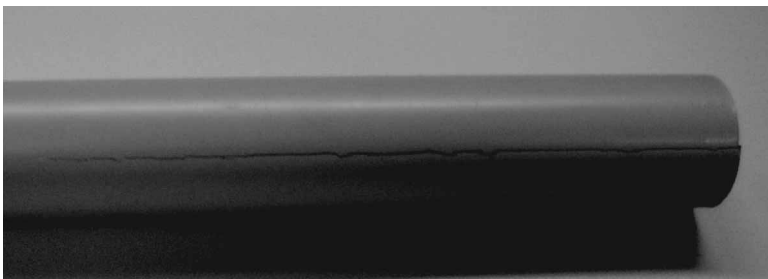


FIG. 8—Longitudinal cracks on a CPVC sprinkler pipe.

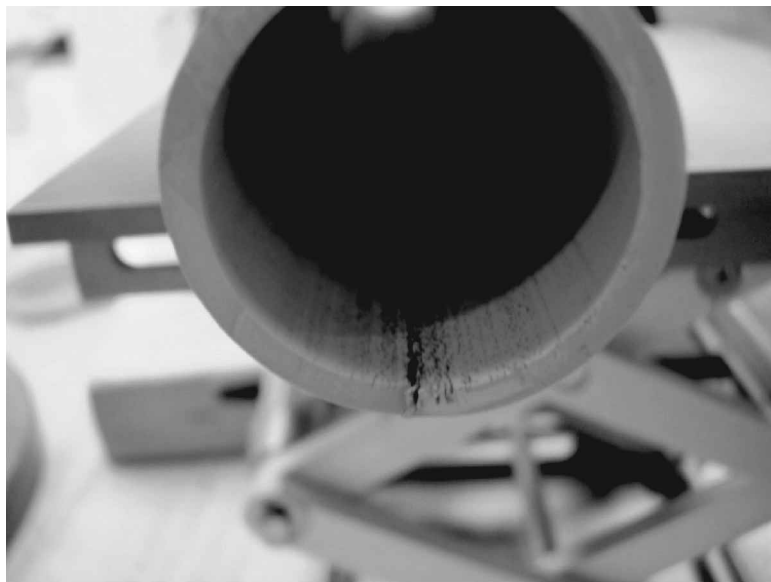


FIG. 9—Secondary cracks and localized darkening on the pipe interior.

wall are crack propagation lines and suggest multiple sites of crack initiation. These lines were formed when adjacent cracks merged.

The fact that several cracks grew parallel to each other and in close vicinity indicates a rapid crack initiation process. It follows that cracks initiated early in the service life of the pipe and grew slowly thereafter, consistent with environment-induced stress cracking. Systems such as sprinkler water supply lines are of particular concern for environmental stress cracking because contaminants can be introduced into the system during installation or service of the mechanical equipment and then remain in the pipe because the water is stagnant. While we did not determine the environmental conditions associated with this system, the oily contaminants on the inner surface of the pipe were suspected to be a contributing factor to pipe failure. Typically, the composition of non-volatile organic solvents can be identified using FTIR [6]. During this particular investigation, all parties agreed to replace the portion of the system where cracks were found without further investigation to determine if the oil contamination or material defect caused the failure.

In contrast with PE compounds, the cell classification for CPVC compound per ASTM D3915 does not include testing for slow crack or environmental stress cracking as part of the requirements [7]. Additionally, the pipe specification ASTM F442 does not require slow crack or environmental crack resistance [8]. Without standardized qualification tests for stress crack resistance, CPVC piping systems are at risk for early failure. CPVC pipes should be tested for all the contaminants that are expected in service.

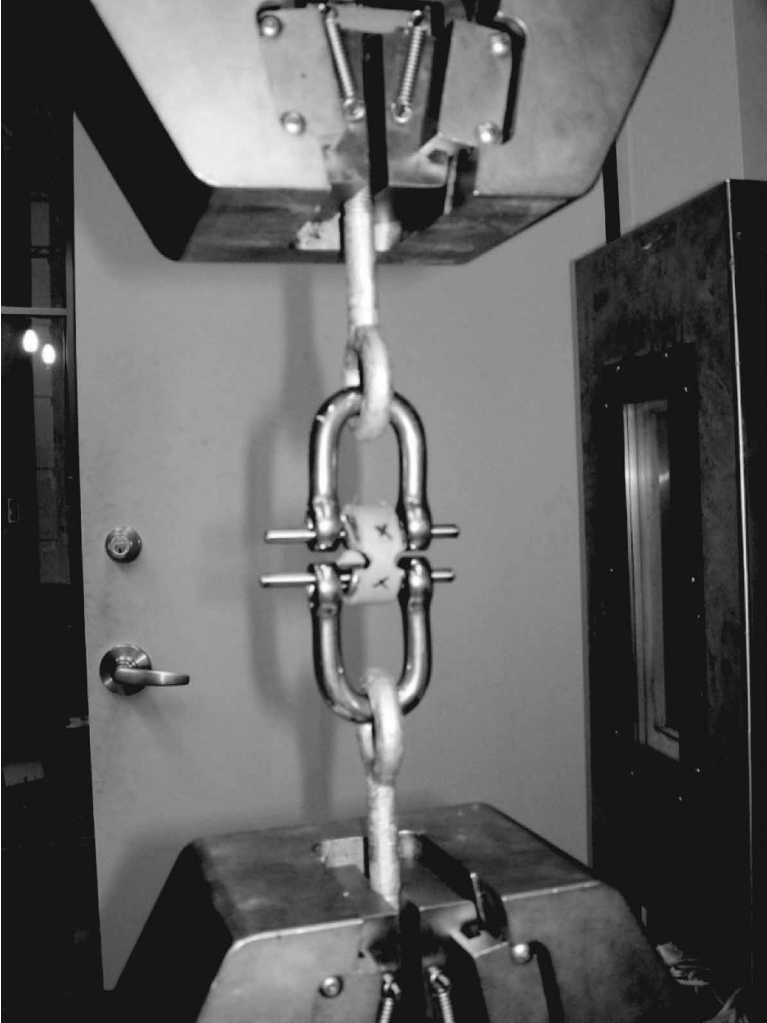


FIG. 10—Split-disk test of initial pipe material for creep resistance.

Implications from this investigation include:

- CPVC can be susceptible to slow crack growth induced by the environment in water even at ambient temperature and with a short exposure time;
- In addition to the normal content in water, common contaminants in non-potable waters can reduce the cracking resistance of CPVC; and
- CPVC materials should be tested for all the conditions and common contaminants that they may be subjected to in practice.

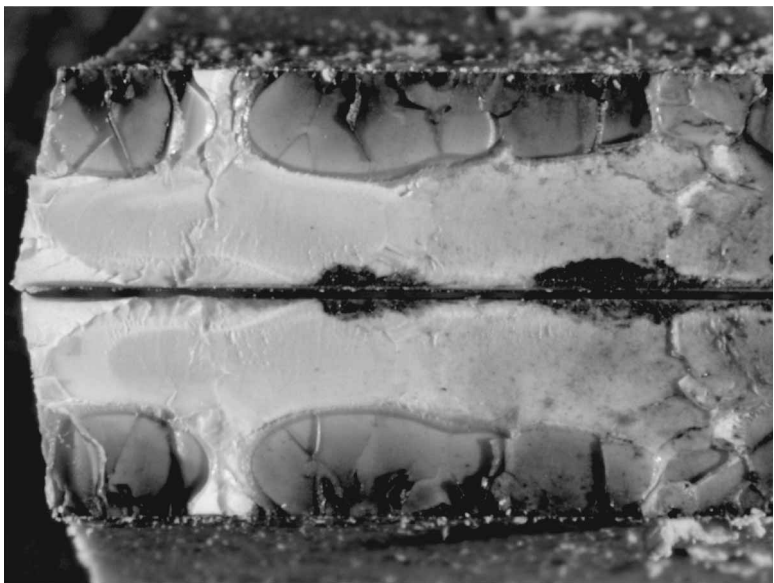


FIG. 11—Matching fractured surfaces of the pipe wall showing the surfaces hinged about the outside surface of the pipe.

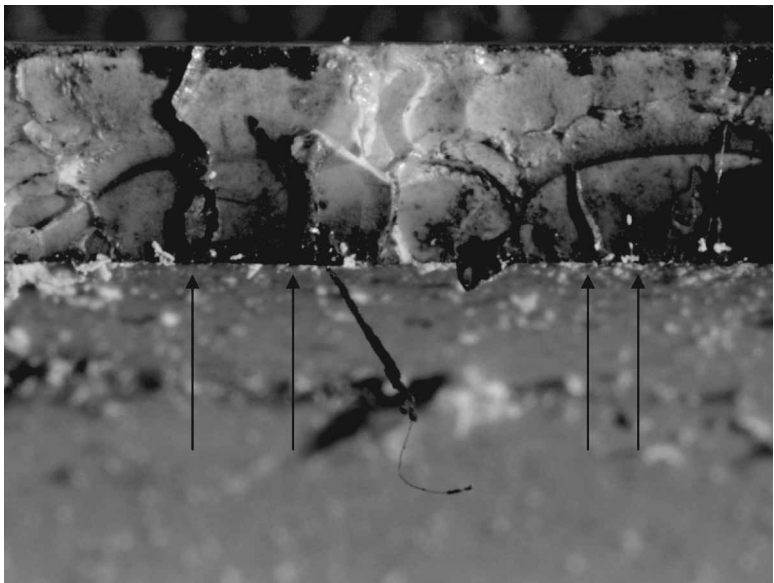


FIG. 12—Fracture surface of the CPVC pipe near initiation sites with arrows pointing to crack propagation lines.

Case Study 3—Over-insertion of PVC Pipe Joint

A buried PVC pressure pipe failed after several years of operation. Excavation revealed a large fracture of a slip-joint bell. Initial inspection of the failed joint revealed that the spigot end of the pipe was inserted beyond the bell shoulder. We were asked to determine if the over-insertion was the sole cause of failure.

As shown in Fig. 13 the spigot end of the pipe had been inserted about 1 in. beyond the full insertion point at the base of the bell shoulder.

Prior experience has shown that over-insertion of slip-joints can result from several installation problems including excessive homing force such as by heavy equipment, lack of a homing mark on field cut pipe, and not ballasting previously installed pipe as successive pipes are homed. ASTM installation standards for thermoplastic pipe provide specific guidance on proper assembly of elastomeric seal slip-joints; see for example ASTM D2321 [9].

At one location around the circumference of the joint the spigot was in tight contact with the bell. Further examination revealed that the bevel on the spigot end at this point was very shallow, suggesting an improperly made field bevel (Fig. 14). The fracture initiated near this point (Fig. 15).

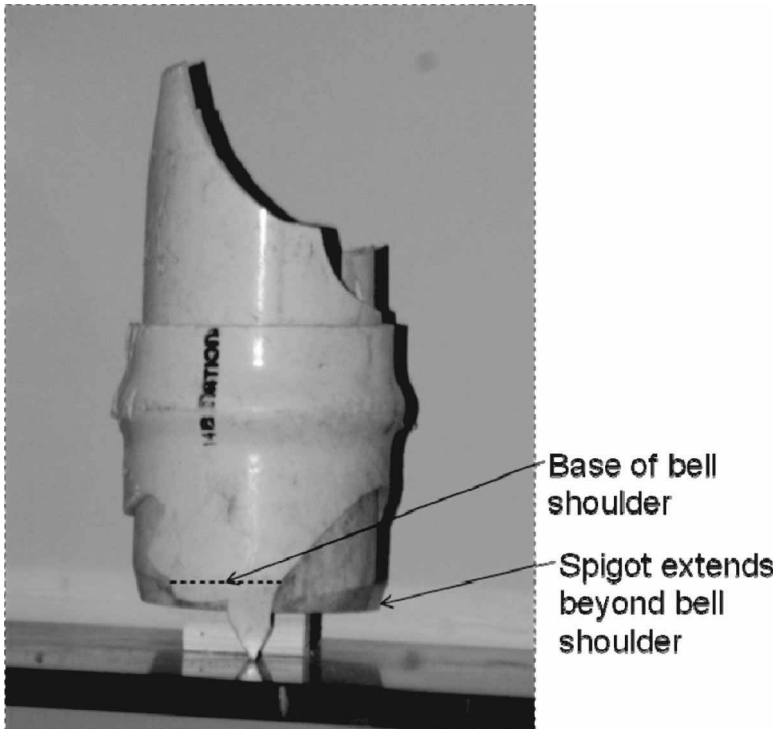


FIG. 13—Fractured bell showing over insertion of spigot.

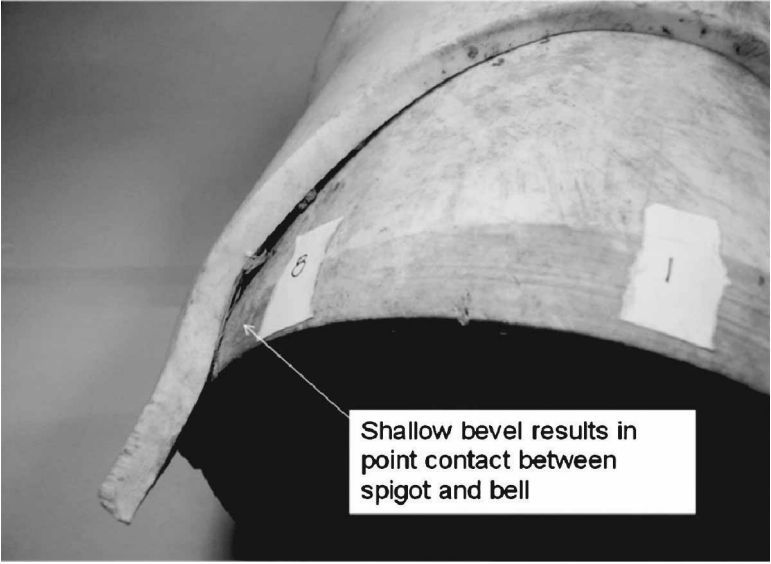


FIG. 14—*Shallow field bevel on spigot caused point contact with bell.*

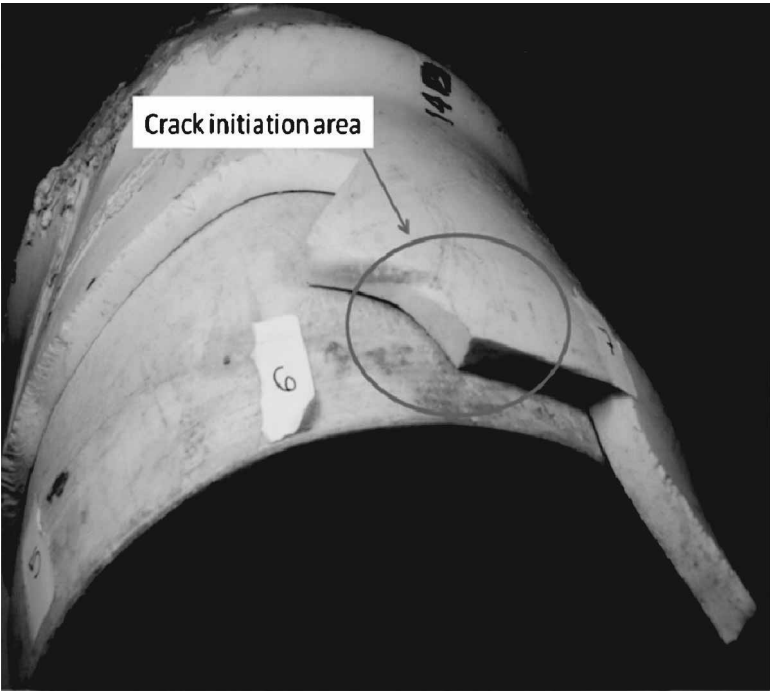


FIG. 15—*Crack initiation area near point contact of field bevel.*

By considering a typical homing force and the friction forces between the spigot and bell, we estimated the circumferential (hoop) stress caused by the eccentric contact force at the location of the shallow bevel to be about 4270 psi (29 MPa) (Fig. 16). This compares to a hoop stress of only about 200 psi (1.4 MPa) for a similarly over-inserted spigot, but with a properly made, uniform bevel. The combination of the high stress due to eccentric contact force and a calculated operating stress of 1990 psi (14 MPa) due to internal pressure and transient forces results in a total hoop stress that approaches the short-term strength of the material, which is 6500 psi (45 MPa). The actual strength capacity of the material after 2 years of service is less than its short-term strength because of the effects of cyclic loading (fatigue) and creep, which have been well-documented to reduce the strength capacity and service life of structural plastics [10,11]. Therefore, failure is predicted to occur well before the design life of the pipe can be attained. The cause of failure was a combination of over-insertion and a poorly made field bevel that resulted in a point force on the bell.

The principal lesson from this investigation is that strict adherence to installation requirements for preparation and assembly of slip-joints is critical to preclude excessive and potentially damaging contact forces. In this case, a combination of over-insertion and an improperly made field bevel resulted in the fracture of the pipe. The failure could have been avoided by following standard installation guidelines such as:

- Placing a homing mark on field cut pipe and ensuring the spigot end is not inserted beyond the mark;

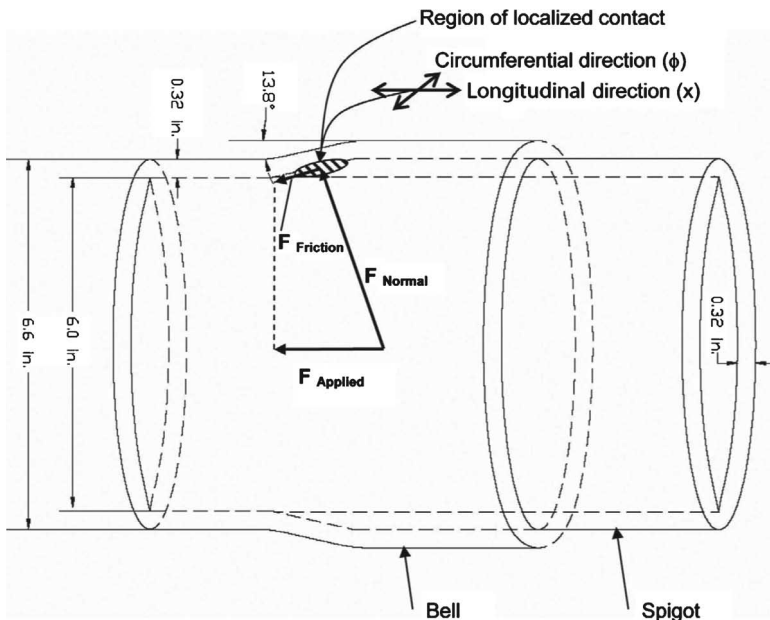


FIG. 16—Forces at concentrated point of contact due to shallow bevel.

TABLE 1—Summary of case studies.

Failure Category	Plastic Product	Key Failure Analysis Technique(s)	Key Finding(s)	Implications
Mechanical	Thermoplastic storm retention units (Case study 1)	• Field investigation • Visual examination • Load analysis • Mechanical testing	• Field investigation indicated excessive lateral deformation at system perimeter and buckled and cracked struts • Analysis suggested low buckling strength of struts • Testing determined actual buckling capacity • The long-term strength of the struts was insufficient to sustain the imposed loads	• Lateral loads must be considered in design • Testing needed to corroborate capacity of critical components • Design must account for long-term creep effects
	Molded HDPE submerged support anchor	• Visual examination • Scanning electron microscope (SEM) • Information review (service loads and material capacity)	• Multiple parallel cracks at the crack initiation site (Fig. 17) • No inclusion or other apparent defect at crack initiation site (Fig. 18) • Cyclic service load exceeded fatigue strength of HDPE • The cyclic fatigue life was inferior to the static creep life.	• The resistance of HDPE to cyclic loads is inferior to its resistance to constant loads

TABLE 1—(Continued).

Failure Category	Plastic Product	Key Failure Analysis Technique(s)	Key Finding(s)	Implications
Environmental Compatibility	PVC gravity sewer pipe	• Visual examination • Mechanical testing • Optical microscopy	• Failure at manufactured seam of pipe • Short-term tensile strength at seam is 65 % of the pipe strength away from the seam • Limited apparent fusion on the failed seams (Fig. 19)	• Seam bond defects compromise long-term strength capacity at seams
	Extruded CPVC pipe for a sprinkler system (Case study 2)	• Visual examinations and measurements • Optical microscopy • Mechanical testing	• Failure away from joints or scratches • Pipe interior has several micro-cracks and cracking progressed slowly from the pipe interior for more than 2/3 of the wall thickness before the final failure • Creep testing revealed creep resistance in agreement with applicable standards	• CPVC pipes can be susceptible to environmental cracking in a water-based application at ambient temperature. • Stress crack testing should be considered, including effects of possible contaminants in sprinkler systems

TABLE 1—(Continued).

Failure Category	Plastic Product	Key Failure Analysis Technique(s)	Key Finding(s)	Implications
Installation	Threaded PVC fitting connecting to storage tank	• Information review • Visual examination and measurements	• Threaded fitting disengaged from the port on an acid storage tank (Fig. 20) • PTFE tape was used • Pressure rating of the fitting is inferior to the rating for other components of the system • Operational temperature exceeded the temperature rating for the fitting	• Creep effects and temperature de-rating of plastics at high temperatures must be accounted for in design
	Bell end of an extruded PVC pipe (Case study 3)	• Visual examinations and measurements • Optical microscopy • Load analysis	• Matching spigot pipe end is over-inserted and with an irregular bevel • High point on the spigot end corresponds to the location of crack initiation in the bell end • Analysis shows high stresses due to point contact at irregular bevel	• Strict adherence to installation requirements for preparation and assembly of slip-joints is critical to preclude excessive and potentially damaging contact forces
	PVC sprinkler pipe	• Visual examinations and measurements • Material testing • Optical microscopy	• Microcracks at the edge of the solvent cement (Fig. 21) • Brittle fracture mode near pipe ID and ductile fracture closer to pipe OD	• Installation procedure and solvent cement characteristics should minimize solvent cement drips beyond the fitting
CPVC pipe and fittings	CPVC pipe and fittings	• Visual examinations and measurements • Material testing • Optical	• Failures concentrated at joints, particularly swing joints • Visual indications of failed joints included bulges and wrinkling on pipe on the exterior and excess solvent cement on the interior (Fig. 22)	• Installation procedure and solvent cement characteristics should minimize solvent cement puddles in the joint

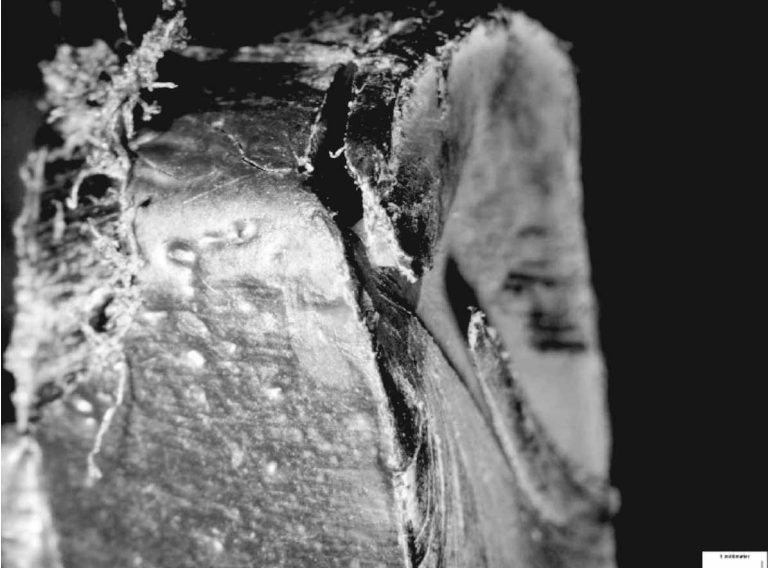


FIG. 17—Initiation site of HDPE support anchor showing parallel cracks.

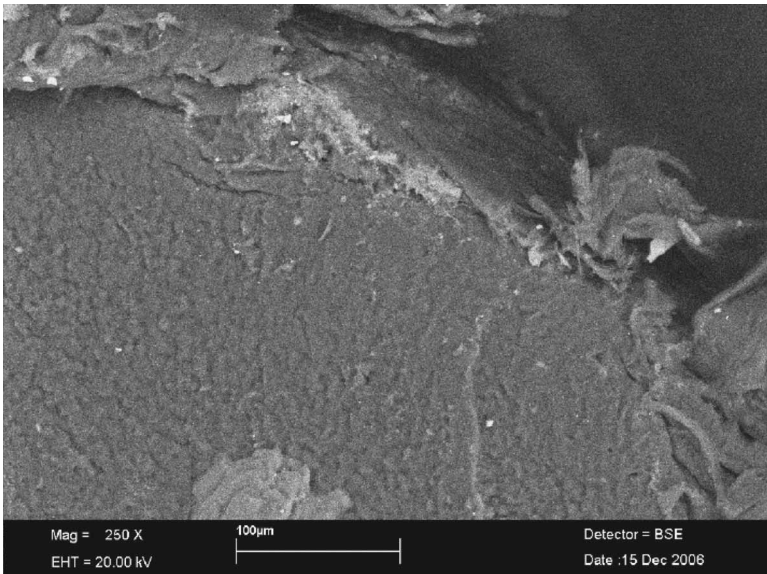


FIG. 18—SEM image of HDPE support anchor showing no inclusions or other defects at site of crack initiation.

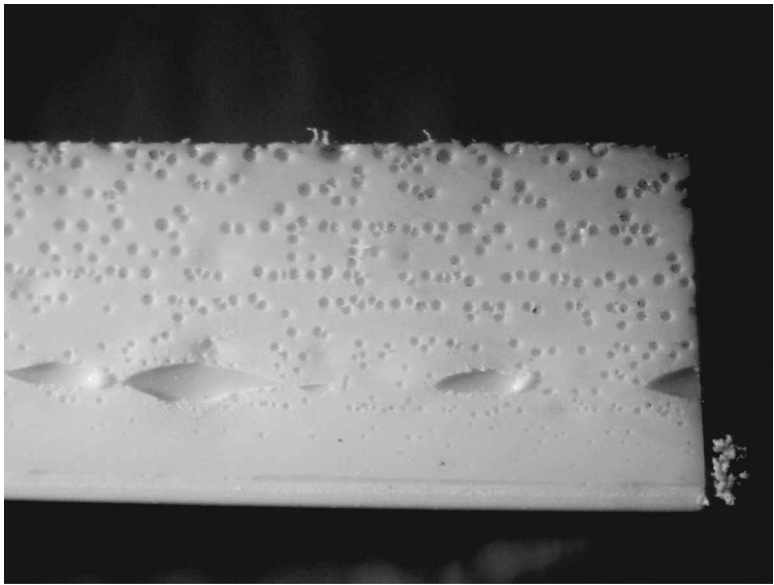


FIG. 19—Cross section of PVC pipe showing high void content at the failed seam.



FIG. 20—Failed connection of threaded PVC fitting to acid storage tank.

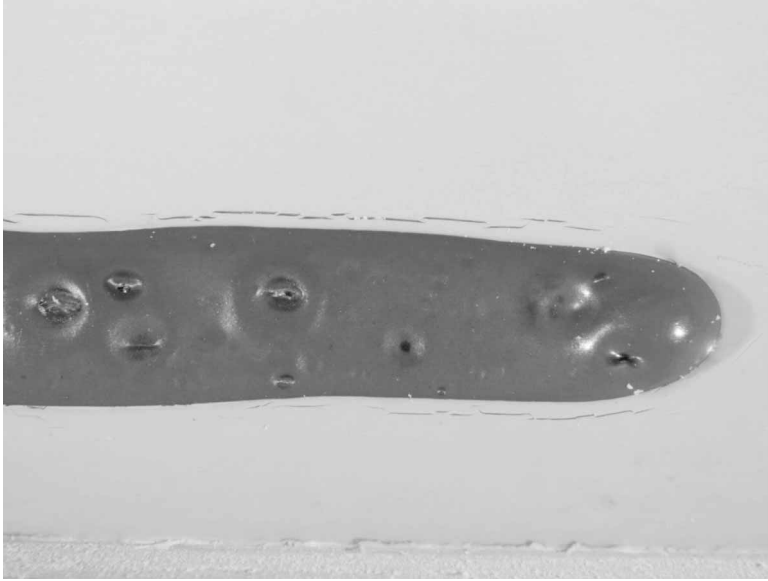


FIG. 21—Microcracks at the edge of a solvent cement drip in a CPVC sprinkler pipe.

- Ballasting pipe after joint assembly to prevent additional movement as successive pipe lengths are installed;
- Measuring and cutting a uniform field bevel on the field cut spigot; and
- Avoiding excessive homing forces by using manufacturer's recommended jointing techniques.

Table 1 presents a summary of the three case studies described above as well as additional case studies. All three cases of mechanical failures involved the long-term strength of the manufactured product. Understanding the effects of sustained load, creep, and fatigue are essential to design for long-term service. The two cases of environmental cracking illustrate the need to consider the nature of the contained fluid as well as the service temperature. The three cases of installation highlight key parameters that require special care in the specification of, and adherence to, installation requirements. As illustrated in the case summaries, problems in the construction of pipe joints are often at the root of installation-related failures.

As shown in Table 1, the key failure analysis techniques can vary from one failure category to the other but common to all the cases presented in Table 1 is visual examination. Careful visual examination, often augmented with microscopy, helps develop a failure hypothesis and an approach to either verify or refute the hypothesis. The review of the information available regarding the service conditions is also important, and is critical for accurately modeling the forces acting on the failed part or structure. Depending on the failure, one or more techniques described in the approach herein may be utilized to determine

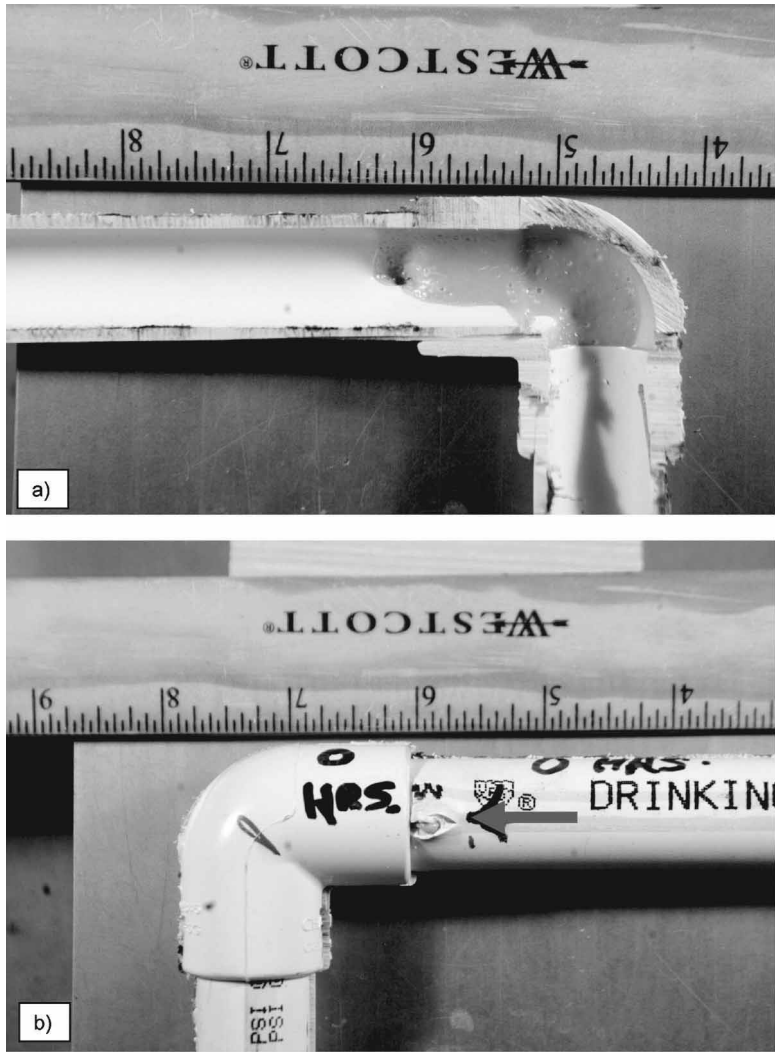


FIG. 22—CPVC pipe with (a) puddle of solvent cement in elbow and (b) a blowout failure of pipe just beyond the fitting during pressure testing.

the cause of the failure. A successful failure analysis requires expertise in all the available techniques as well as experienced judgment in implementing the appropriate ones and interpreting the results.

Concluding Remarks

The case studies presented in this paper provide examples of mechanical failures, environmental attack, and installation defects. Failure analysis based on a few or several of the tools summarized herein can help determine the proximate

cause of the failure. The determination of the conditions and processes that led to failure provides valuable insight into the performance limitations and attributes of structural plastics for pipe and other hydraulic structures. Lessons learned from these failures ultimately enhance engineering knowledge and lead to improvements in design, manufacture, and construction practices.

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Challenges in Investigating Chlorinated Polyvinyl Chloride Pipe Failures*

ABSTRACT: Chlorinated polyvinyl chloride (CPVC) is increasingly replacing steel pipe in fire protection systems due to lower cost and ease of installation. However, CPVC is susceptible to failure by mechanisms other than those typically experienced with steel pipe. The losses and remediation costs associated with a simple pipe failure can be extraordinary, as a failed pipe can cause significant water damage to a high-rise building. Identifying the cause(s) of pipe failure can be challenging for a number of reasons. First, CPVC pipes can fail by several mechanisms, including high pressure, impact, environmental stress cracking (ESC), and manufacturing defects. ESC can be particularly vexing during a failure investigation. Potential ESC agents include many lower molecular weight and highly mobile compounds, which are soluble in the polymer. Common construction compounds such as adhesives, plumber's putty, fire barrier caulks, cutting oils, and glycol-based anti-freezes are potential ESC agents. Second, the situation is complicated by the fact that only trace amounts of a contaminant may be necessary to cause ESC. The pipe failure event also tends to flush out the contaminants, further reducing their concentration on the pipe surface. What little contaminant remains can be difficult to identify or "fingerprint." In addition, pipe defects including incomplete fusion, thermal damage, excessive residual stresses, or irregular molecular weight distributions may also contribute to ESC failures.

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Yet, they are not always apparent or easy to detect. This paper explores the challenges encountered during numerous failure investigations and reviews the state of the art in testing and analysis as applied to ESC in CPVC pipes.

KEYWORDS: CPVC, pipe, environmental stress cracking (ESC)

Background

Polyvinyl chloride (PVC) was invented in the late 1800s, and it first found large-scale use as fire resistant wire insulation on U.S. Navy ships during Word War II. By the 1940s, plasticized PVC had found its way into various consumer products. In the 1950s, extrusion and molding of rigid PVC were developed for the manufacture of pipe and fittings. By 2001, PVC served 69 % of the construction market for plastics, primarily in the areas of wire and cable and coatings, in addition to piping. Piping applications include water service distribution, “drain, waste, and vent,” sewer, telephone duct, and fire protection systems [1].

In the case of hot water distribution and fire protection systems, a modified version is preferred: Chlorinated PVC (CPVC). CPVC is produced by a post-chlorination process that increases the chlorine content from 57.4 % in PVC to as much as 70 %. This modification increases the glass transition temperature and improves mechanical properties at elevated temperatures. The additional chlorine content also imparts improved flame and smoke characteristics. Furthermore, CPVC demonstrates good chemical resistance at elevated temperatures, so it has found use for handling of corrosive fluids, including some acids. The manufacture of CPVC dates back to 1928, but widespread use of CPVC did not occur until after the 1950s, when BFGoodrich developed a process for

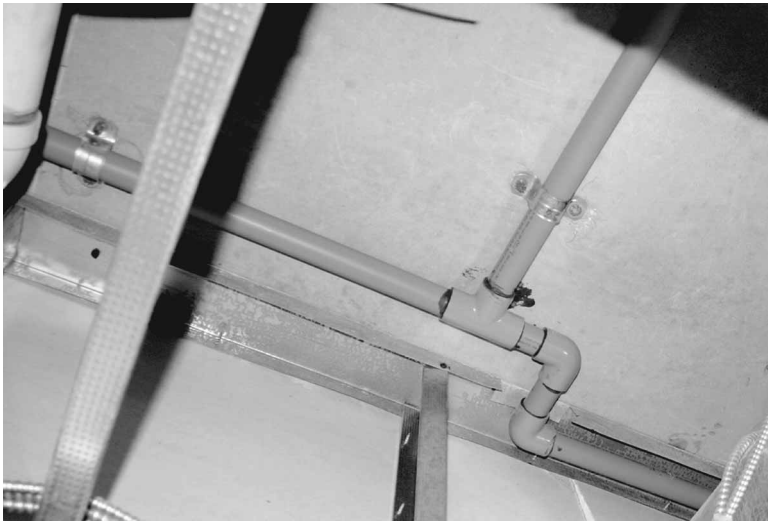


FIG. 1—CPVC pipe installation in a fire protection system.

extruding CPVC pipe [1]. Today, CPVC is widely used in fire protection systems due to its low installation costs (Fig. 1). CPVC's corrosion resistance is also touted as a key advantage over steel pipe, but, nevertheless, CPVC pipe is susceptible to environmental attack.

Failure Modes and Fractography

CPVC pipe can fail by overload, impact, thermal damage, ultraviolet (UV) damage, and environmental attack. A careful examination of a failed pipe and any fracture surfaces will usually lead to identification of the particular failure mode and help determine the ultimate cause of failure. Fractography, the study of details on fracture surfaces, is a very powerful technique used by metallurgists, materials experts, and other forensic engineers to characterize the manufacturing, loading, and environment history of the fractured part.

First, the location and directionality of cracks can help indicate the type of loading that may have contributed to the failure. When pressurized internally, the stress along the circumferential direction of a pipe ("hoop" stress) is twice as large as the stress along the axial direction, so cracks tend to form along the length of a pipe (normal to the maximum principal stresses). However, the stress distribution can become more complex at joints (where the interference between the pipe and a coupling, elbow, or other adapter introduces additional stress) or at hangers (where the weight of a suspended pipe can also introduce bending stresses). According to elasticity theory, the stress in a pressurized pipe is highest on the inside surface [2], so cracks caused by excess pressure will normally tend to grow from the inside out. Impact loads will tend to induce bending of the pipe wall such that the inner surface experiences higher tensile stress. On the other hand, bending loads due to installation induce tensile stress on the outside surface of the pipe. In the case of environmental attack, cracking can initiate on either surface, depending upon the location of the exposure.

Second, the microscopic appearance of the crack(s) indicates the failure mechanism. For instance, a brittle fracture that occurs at a nominal stress level below the yield strength of the plastic will produce a smooth, dull appearance on the fracture surface in the vicinity of the crack origin (Fig. 2). On the other hand, a smooth, glassy fracture morphology is indicative of slow crack growth and is usually associated with environmental stress cracking (ESC) [3]. An example of ESC crack growth is shown in Fig. 3.

Brittle cracks may also form due to impact or a stress concentration at a defect. Subsequent growth of the crack due to fatigue cycling will result in striation marks, as shown in Fig. 4. Fast fracture of the remaining wall thickness yields a surface morphology that is rough and uneven. This morphology is attributed to high-speed stress waves propagating along the crack plane [4]. Ductile overload that is not preceded by brittle crack growth may occur due to an over pressurization or freezing event. In this case, the surface morphology will exhibit a relatively rough texture indicating microductility. For example, Fig. 5 shows the fracture surface of a CPVC pipe that failed due to excess pressure resulting from a freezing event.

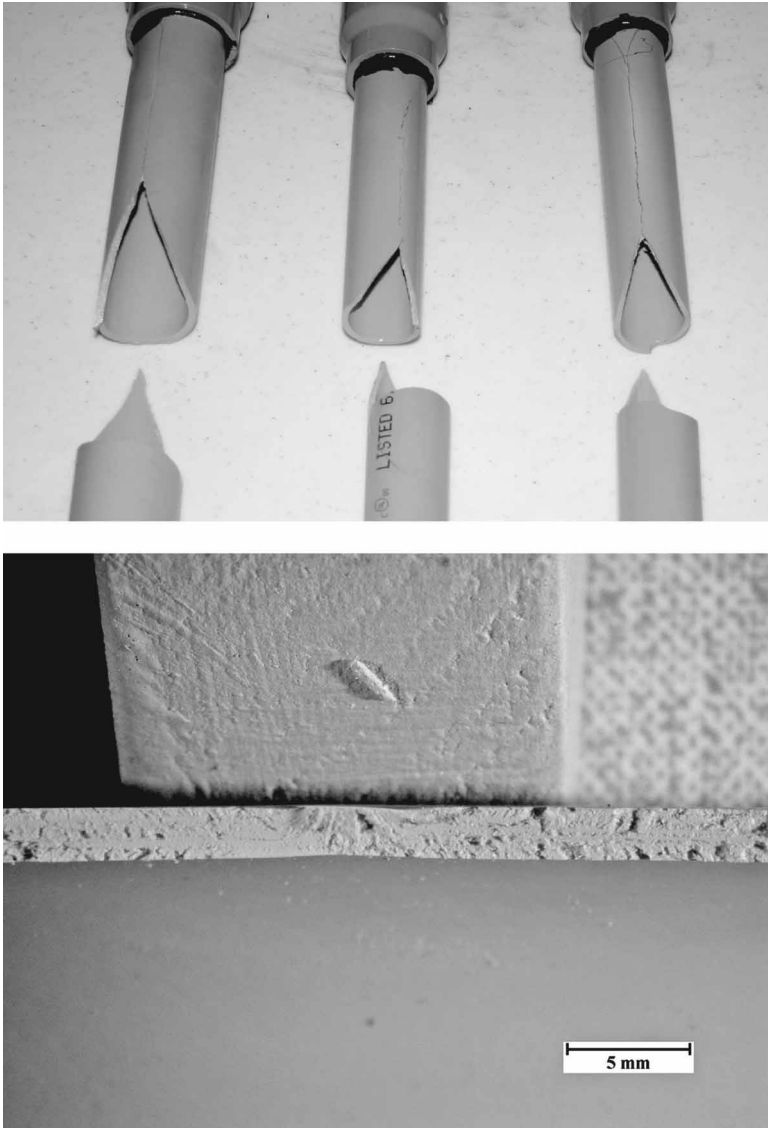


FIG. 2—Mechanical overstress failures in CPVC pipe.

Plastic pipe is designed to leak before breaking so that a single, relatively short crack can pervade through the entire pipe wall thickness and reach the outer surface without causing gross pipe failure. By selecting an appropriate wall thickness to diameter ratio (termed “standard pipe dimension ratio” or SDR) and providing sufficient fracture toughness through compounding and processing, the manufacturer can ensure that the pipe should always leak before bursting. Thus, the occurrence of ductile failures is low.

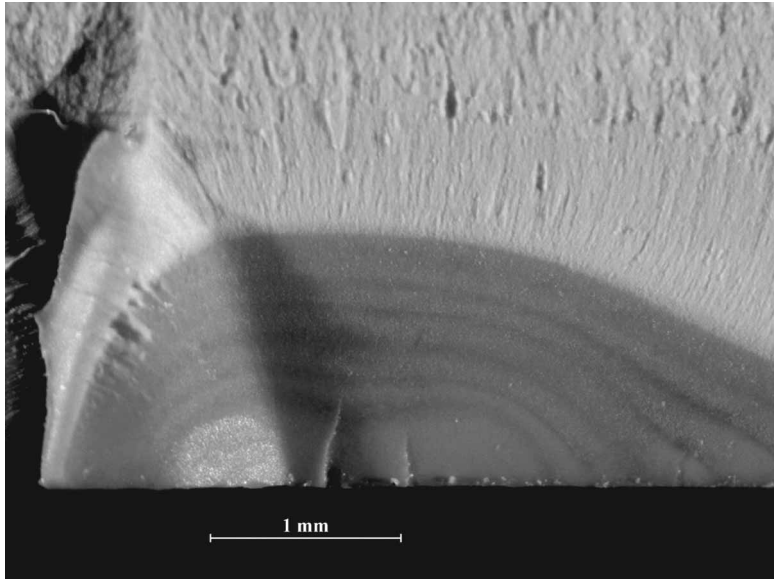


FIG. 3—ESC due to polyethylene glycol exposure.

On rare occasions, defects may occur in an extruded pipe due to improper processing. Such defects include weld or knit lines [5,6], contaminants [6], and thermal and mechanical degradation [7]. Extruded plastic pipe contains

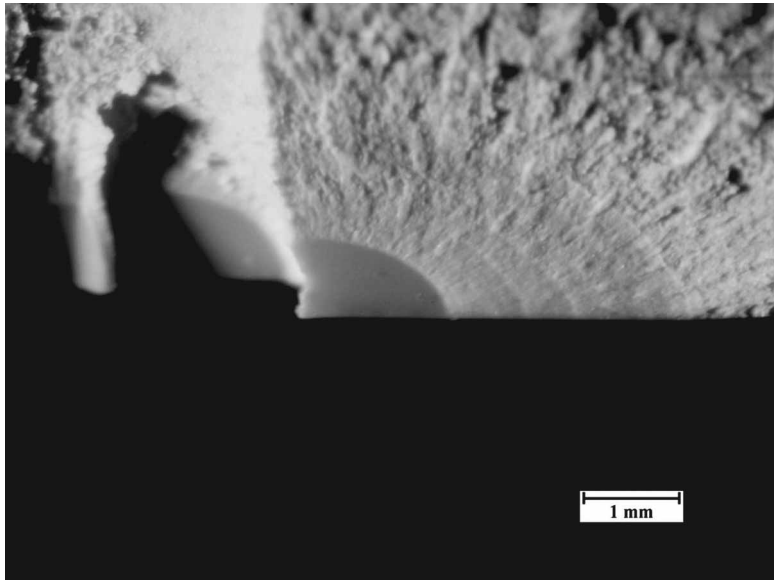


FIG. 4—Fatigue striations after the introduction of an ESC crack.

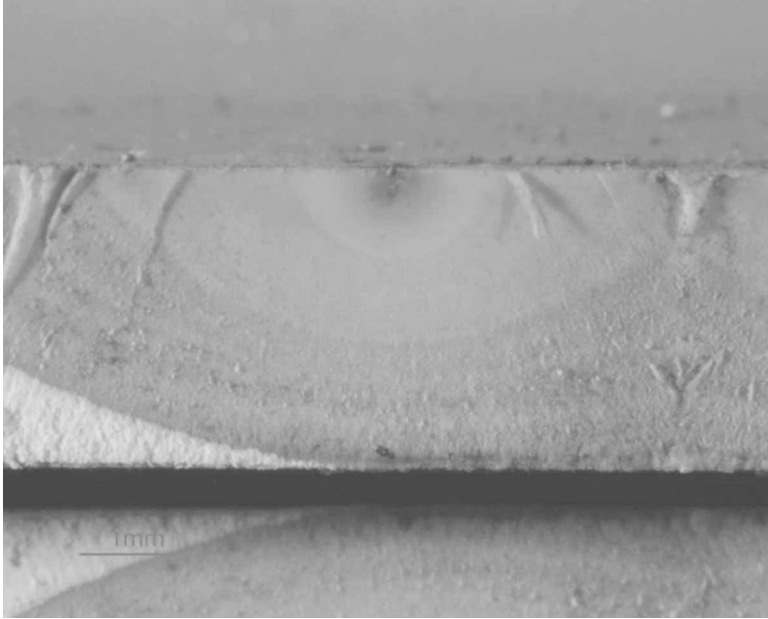


FIG. 5—Fracture surface of a crack in a CPVC pipe that initiated as a result of a freezing event.

longitudinal “knit” lines or “weld” lines, which result when the melt polymer separates and then rejoins as it flows around the inner mandrel supports called “spider legs” [6]. If the process conditions are not properly controlled, the polymer may not completely fuse at these interfaces leaving them weak and possibly susceptible to penetration by an ESC agent [8].

Thermal and UV Damage

Thermal and mechanical damage during processing can cause chain scission, resulting in a reduction in molecular weight. Specifically, thermal oxidative degradation of PVC or CPVC results in loss of HCl (dehydrochlorination) and formation of a double bond between carbon atoms in the polymer backbone [1,9,10]. Following thermally induced dehydrochlorination, CPVC pipe may also be susceptible to photo-oxidation via UV radiation between wavelengths of 300 and 400 nm, resulting in the formation of carboxylates and hydroxyl groups [10]. Photo-oxidation of PVC or CPVC (and the subsequent chain scission and crosslinking) results in loss of gloss, discoloration (yellowing then whitening), chalking, formation of cracks, and surface embrittlement [10]. Surface embrittlement occurs only within the first 200–400 μm of exposed surface [11], but as cracks form on the surface, newly exposed material will also degrade and this can eventually extend deeper into the sample.

Beyond visual observation, thermal and UV damage can be detected by several different techniques. First, thermally degraded PVC becomes strongly fluorescent in the visible region when irradiated with UV. Second, several spectrographic methods may also be used to detect thermal/UV damage, including UV, Fourier transform infrared (FTIR) spectroscopy, and Raman spectroscopy. UV spectroscopy can be used to detect reflectance of a sample in the 600–700 nm range, where newly formed polyene sequences (conjugated double bonds) will absorb. FTIR can be used to detect carbonyls (C=O bonds) and polyenes (C=C) with key peaks near 1712 and 1660 cm^{-1} , respectively. Comparing these peaks to a peak at 1430 or 1330 cm^{-1} allows for determining changes in concentration of the carboxylates.

If there is no difference in concentration of carbonyls in the interior of a sample and there are polyene sequences throughout the sample, it indicates that the PVC was most probably thermally degraded during processing. If there are more carbonyl groups near the surface, it is an indication of UV degradation and photo-oxidation. Many additives used in PVC compounds have FTIR peaks in the same region as carbonyls and polyenes. Thus, there are specific test procedures to follow so that additives in the PVC compound do not interfere with the test and interpretation [11,12]. Nevertheless, interference from the additives may preclude the use of this analysis.

Environmental Attack

The presence of certain contaminants can also promote failure of PVC and CPVC pipes. Highly concentrated solutions at elevated temperature can cause surface damage, despite CPVC's high resistance to acids and caustics. In contrast to PVC, CPVC is also susceptible to dehydrochlorination by ammonia, ammonium hydroxide, and most amines [3]. However, the most common mode of environmental attack in CPVC pipe is likely ESC. ESC is the failure of a polymer under applied tensile stress and in the localized presence of a contaminant or ESC "agent." Like creep, ESC can occur at stress levels well below the design stress for the part. (In fact, some view creep, time-dependent deformation under constant stress, as a special case of ESC that occurs in air.) ESC is often likened to stress corrosion cracking in metals, but, while the processes may be analogous, they are very different failure mechanisms.

Generally, ESC occurs when a substance is able to both wet the surface of the plastic and also diffuse into the polymer network [13]. The degree of absorption may determine the likelihood of causing ESC. The ESC agent then acts as a plasticizer, increasing the mobility of the polymer chains. ESC is not a chemical attack; no chemical reactions occur. Rather, ESC agents interfere with the intermolecular (weak) forces that bind the polymer chains to together, accelerating molecular disentanglement. ESC agents also accelerate the process of brittle crack formation through crazing. The ESC agent penetrates a craze tip and plasticizes (softens) the fibrils that bridge the craze, allowing the craze to grow. The fluid also exerts a pressure on the craze, acting like a wedge.

On a macroscopic scale, the ESC cracks often appear as series of interconnected cracks emanating from the contaminated surface at a number of different initiation sites. Sometimes only a single crack will form, causing an isolated “pin hole” type leak. Typically, however, a large number of circumferential cracks will develop, forming what is often referred to as a “mud crack” pattern. As the cracks grow, they can coalesce and form a single long crack that results in gross failure along the length of the pipe [8,14]. Thus, a pipe that is designed to leak before bursting may actually burst as a result of ESC.

Amorphous polymers like CPVC are more prone to ESC than semi-crystalline polymers [13]. They have more free volume due to their less-ordered morphology, which allows for greater infiltration by the ESC agent. ESC resistance (ESCR) increases with higher molecular weight due to increased molecular entanglement. Like creep, a visco-elastic mechanism, the rate of the ESC process is temperature dependent, and ESC is enhanced as the temperature approaches the glass transition temperature of the polymer, T_g . ESC is also accelerated by residual stresses, stress concentrations, and dilatational stress.

ESC agents are generally thought to be any species that can be absorbed by the polymer significantly in a short period of time. Naturally, these include plasticizers and solvents. The solubility of the ESC agent within the polymer determines its aggressiveness. The rate of diffusion and wettability are also important. In general, most liquids with weak to moderate hydrogen bonding can be ESC agents, depending on the plastic: Aromatic hydrocarbons, halogenated hydrocarbons, ethers, ketones, aldehydes, esters, and nitrogen and sulfur containing compounds. Aliphatic hydrocarbons and liquids with strong hydrogen bonds, such as water and alcohols, are less aggressive. Liquids with high molar volume are also less likely to be ESC agents due to their high viscosity and boiling points. ESC agents with lower molecular weight are more aggressive (due to greater mobility). ESC agents are more aggressive at temperatures near their boiling point [13].

In the case of CPVC, however, there does not appear to be great correlation with solubility or hydrogen bonding of the ESC agent [13]. Experience has shown that various caulks, sealants, fire barrier products, leak detector compounds, and low molecular weight hydrocarbons such as vegetable oils and lubricants may be deleterious [15]. In addition, glycol-based antifreezes can cause ESC.

Challenges in Failure Investigations

Interpreting Details of Material Evidence

In the authors' experience, investigations of pipes failures can be encumbered by a number of factors that are beyond the control of the engineer. For example, the forensic engineer may not be retained until after the initial examination of the pipe by the fire protection company, building owner, or insurance company. In a rush to return the system to service, a failed pipe is often cut out and replaced without any regard for properly preserving the evidence, which may

include adjacent materials (i.e., potential contaminants), water, and the pipe itself. In addition, the location and orientation of the failed pipe, the location of hanger supports and other potential loads or constraints, and general history of the pipe may not be properly documented.

Fortunately, a great deal of information can still be ascertained by examination of the failed pipe and its fracture surfaces. Yet, inexperienced investigators may sometimes misinterpret the fractographic features by

- Confusing brittle cracks caused by mechanical damage (smooth, dull) with ESC cracks (smooth, glassy),
- Misinterpreting crack propagation directions, as evidenced by crack branching,
- Attributing failure to secondary damage mechanisms, or
- Ignoring loading history.

Consider again the images in Figs. 3 and 5. Both images were taken from CPVC pipes that failed due to slow crack growth originating at multiple locations, and both fracture surfaces are characterized by smooth crack origins and “beach” marks showing multiple crack growth and arrest cycles. However, the first image shows a dull appearance of the initiation area and microductility in the growth region, whereas the second image shows a glassy appearance in the initiation area and throughout the crack growth region, characteristic of ESC. The microductility in the former sample’s crack growth region is indicative of a mechanically driven crack, not ESC. This particular pipe likely failed due to a water freezing event that resulted in excess pressure and initiated a crack, followed by elevated temperatures, which helped to propagate the crack.

In addition, it is not uncommon for investigators to attribute failure to secondary damage. This often occurs when the fractographic detail or loading history is neglected. For instance, in one case, a party alleged that a CPVC pipe failed due to UV damage as a result of improper storage (Fig. 6). The outer surface of the pipe was slightly discolored, and FTIR analysis of the damaged surface indicated some degradation (Fig. 7). However, as shown in Fig. 8, the pipe actually failed by a crack that grew from the inside surface outwards, and it appeared to initiate at a spider/knit line. The crack also demonstrates the glassy, smooth morphology of ESC. Nevertheless, it was alleged that the UV damage compromised the integrity of the pipe, making it more susceptible to environmental attack. In this case, though, the UV damage was limited to the outer surface.

Applying and Interpreting Materials Analysis and Test Methods

ESC failures can prove particularly challenging in failure investigations, as identifying the contaminant can be difficult. This is typically done using FTIR or gas chromatography/mass spectroscopy. Sometimes the ESC agent is easily identified, but oftentimes the identification can be complicated by a number of factors. First, many times, only trace amounts of the contaminant remain after a pipe failure. This may be due to the initial low concentration of the contaminant, or it may be a result of the pipe and fracture surfaces being flushed out during the failure event. Furthermore, the spectrographic methods only provide

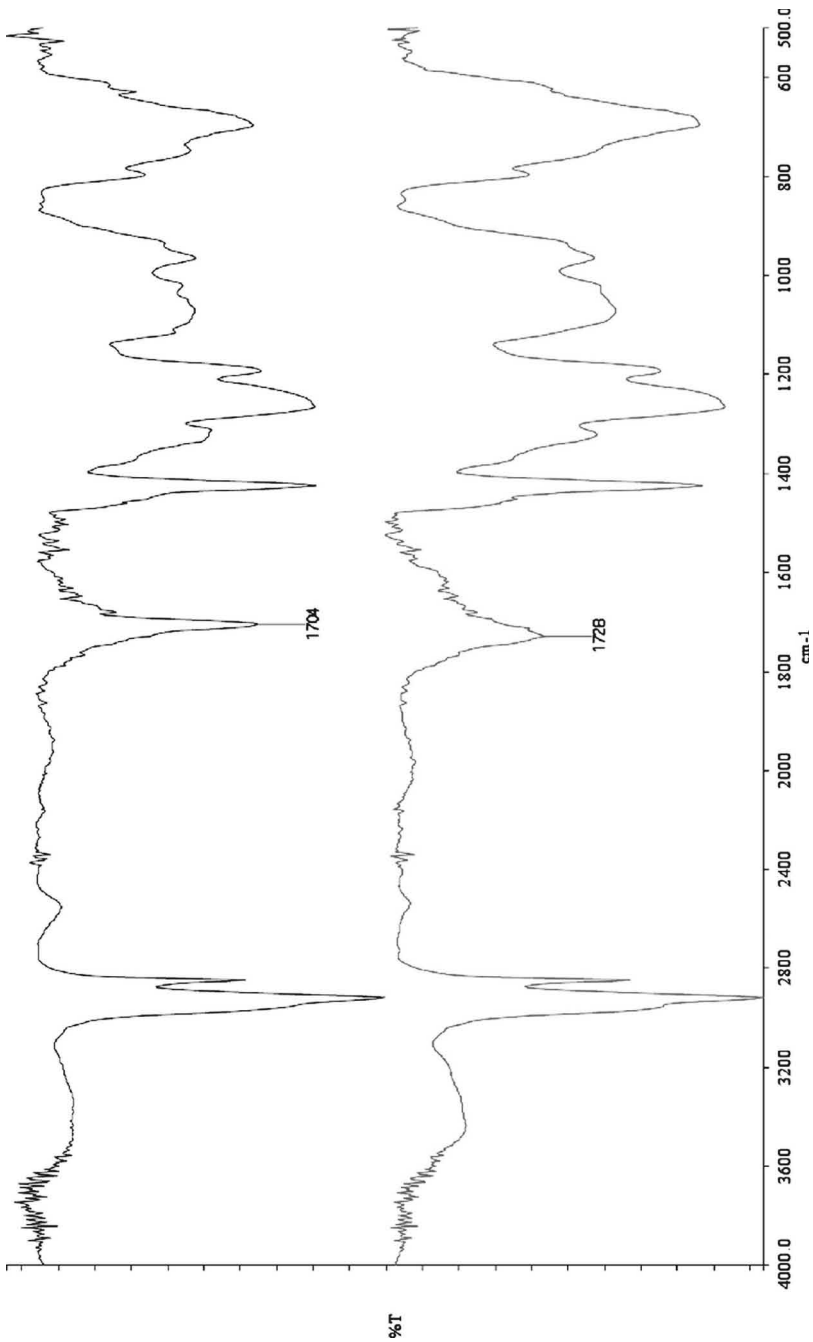


FIG. 6—Evidence of UV damage on the outer surface of a CPVC pipe: Note significant difference in carbonyl peak at around 1720 cm^{-1} between inner surface (top) and outer surface (bottom).

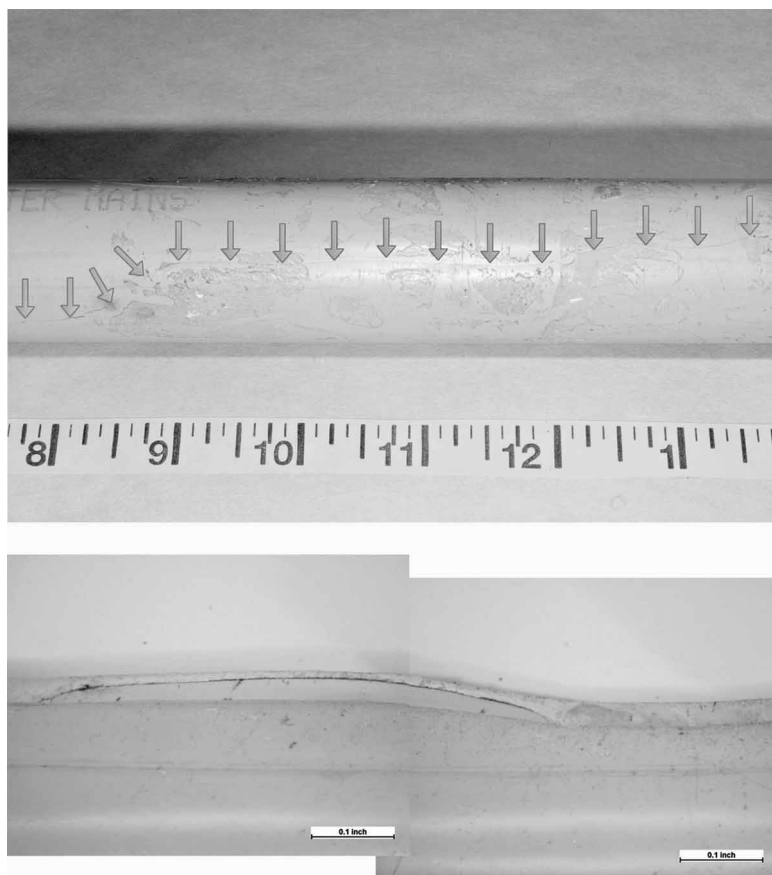


FIG. 7—A failure resulting from a single ESC crack at a spider line.

probable matches to standard spectra in library data. Without a reference material for comparison, definitive matches can be difficult to obtain, especially since, without knowledge of a CPVC compound formulation or “recipe,” it may be difficult to distinguish between constituents of the CPVC compound and foreign materials. Finally, the chemical “signature” for the ESC agent can be masked by that of the base polymer. This is especially true for low molecular weight hydrocarbons, such as oils.

Further complicating the matter is the fact that the resistance of a plastic to ESC can be reduced by defects or processing irregularities. The presence of spider/knit lines may serve as stress concentrations and promote ESC. The ESCR of polymers is also reduced with lower molecular weight, so chain scission resulting from thermo-mechanical damage during processing can also lead to crack development. Improper heat settings during extrusion can also lead to residual stress. Thus, the final fracture of a pipe may be influenced by multiple,

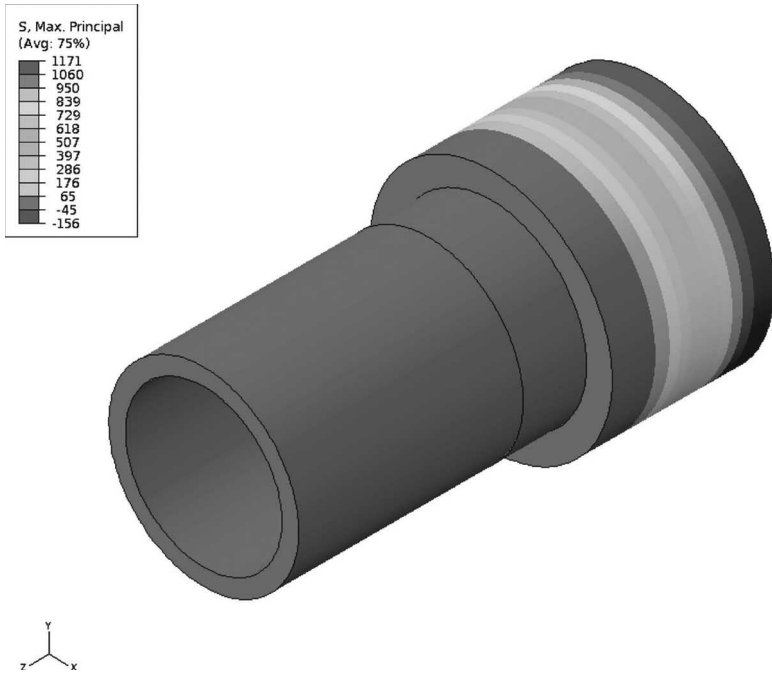


FIG. 8—Stress distribution at the connection between a 1 in. SDR13.5 CPVC pipe (left) and a 1 in. Sch 80 fitting (right). Actual dimensions were chosen at high/low end of tolerance specifications to cause greatest interference fit (half z-symmetry model shown).

synergistic factors. One of the biggest challenges facing an investigator in this situation is to assess the relative influences of the contaminant, material variabilities, and installation details, which may tend to increase the stress on the pipe. The public domain literature is sorely lacking in data to establish critical stress/strain levels, critical ESC agent concentrations, and critical material parameters such as molecular weight and residual stress.

Experimental Testing

In the course of such investigations, it is desirable to have a test method that will allow the investigator to test the quality or ESCR of the subject pipe in question or, alternately, to test the ESCR of the general class of plastic pipe to a suspected ESC agent. However, no standardized ESC test for pipe currently exists. Typically, plastic pipe investigations have relied on a pressurized pipe method, following the general procedure of the ASTM D1598 burst test. This test is certainly most realistic, but it consumes large quantities of pipe and may not be practical for subject pipe removed from a failed system. In addition, for studies involving contaminants on the inside of the pipe, damage cannot be observed until the crack (or cracks) penetrates the entire wall thickness and cause leaking or rupturing of the pipe.

Several test methods are available for flat plastic samples, including the three-point bending fixed strain test [13], the ASTM D1693-08 bent strip test [16], the ISO 22088-3 bent strip test [17,18], the ISO 22088-2 constant tensile stress method [19], and the ASTM F442-99 flattening test [20]. These tests are more useful for experimental purposes due to the small specimen size. Yet, while the three-point bending test, bent strip test, and flattening test all indicate ESC sensitivity by surface cracking, they do not provide any quantitative results, such as the time to failure, stress dependence, and temperature dependence. These tests also enforce fixed displacements/strains, as opposed to loads/stress, and crack growth will tend to reduce surface strains, thereby retarding failure times [13]. On the other hand, these tests are ideal for ESC screening, as the strain level is often chosen to be higher than the service level and cracks may result in a very short time (between 1 day and 1 week). (However, the investigator must be aware that increasing the stress/strain level may change the failure mode from brittle to ductile.)

The constant tensile stress method (ISO 22088-2) is particularly useful in determining the effect of a chemical on the pipe material. Though difficult, specimens can sometimes be machined from the pipe wall and tested with the chemicals of interest in order to measure the effect on failure time. This can be used to establish a stress rating for the material in the presence of the chemical agent.

For situations in which CPVC pipe is removed from a failed system, using any of these methods to assess the quality of the pipe can be difficult, as the results may be influenced by the presence of existing ESC cracks. Screening the potential test samples for cracks is difficult, as ESC cracks may not be visible even under moderate magnification. It would be useful then to have a test method that allows the investigator to stress and expose the surface of the pipe opposite of the initial exposure surface. Thus, a pipe that developed cracks on the inside surface could be tested in such a way that the ESCR of the outer surface only was evaluated. Research efforts in this area are ongoing.

Systems Analysis

In addition to the materials analysis outlined above, a thorough failure investigation often requires a complete fire sprinkler system analysis. To this end, it is constructive to understand the history of the system, including installation procedures, pressure loading, maintenance, and repair. Regarding installation, NFPA 13, "Standard for the Installation of Sprinkler Systems," [21] requires that underground piping, from the water supply to the sprinkler system riser, and lead-in connections to the sprinkler system riser be completely flushed before connection is made to downstream piping. Provisions must also be made to facilitate flushing of sprinkler systems. The intent of these requirements is to remove foreign material that may obstruct or block flow to an individual sprinkler, not to remove ESC contaminants. Thus, potential ESC agents inside the pipe are not necessarily purged from the system prior to service. As a result, sprinkler installers must take extra care to ensure cutting oils used when threading steel pipe are removed prior to installing in systems also utilizing CPVC.

To understand load history, one must consider the municipal water source, as well as the internal workings of the system. Municipal and private water suppliers use pumps and pumping stations to boost pressures in supply mains to (1) ensure that adequate supply and pressure are available to support fire fighting and fire suppression systems, (2) to meet the needs of high-rise buildings where pressure is lost as the elevation increases, and (3) to maintain water supply in water towers and tanks. It is not uncommon for pressure in water supply mains to exceed 200 psi, which may exceed the maximum working pressure of CPVC pipe.

Most plumbing codes require pressure-reducing valves on domestic systems where the water main pressure exceeds 80 psi. Pressure-reducing valves ensure a constant flow of water at a set pressure. A pressure-reducing valve not only regulates pressure but also typically absorbs fluctuations in pressure that can be both of high magnitude and high frequency. However, municipal and private water suppliers often increase supply pressures over time as communities grow and the number of water users increase. Hence, it is not uncommon to find plumbing systems without pressure-reducing valve protection operating above 80 psi because one was “not required” at the time of initial construction.

Once systemic failures such as those resulting from ESC begin to occur in a building, few options exist outside of full mitigation. One potential means to slow the rate of failures is to reduce the pressure, thereby reducing the stress on the pipe and fittings. However, the stress level (or threshold) below which ESC does not occur is unknown. Furthermore, even in an unpressurized system, stresses still exist at connections and hangers and possibly due to residual stresses in the pipe. The installation related stresses depend on the quality of installation (i.e., hanger spacing) and variabilities in the pipes and fittings. For instance, consider the case of a 1 in. diameter SDR13.5 CPVC pipe inserted into a 1 in. ch Schedule 80 coupling. The coupling is tapered such that the inner diameter is nominally 0.015 in. less at its middle than at the free ends so that an interference connection is formed as the pipe is inserted into the coupling. This connection has the benefit of ensuring a uniform bond line when the solvent cement is introduced, and it promotes intimate contact between pipe and fitting.

However, the interference fit also generates a circumferential stress (and a small axial stress) on the outer surface of the fitting. If the pipe diameter and thickness are at the high end of the allowable tolerances, as specified in ASTM F442-99 [20], and the internal diameter of the fitting is at the low end of its tolerance, as specified in ASTM F439-06 [22], then the initial stress can be as large as about 1200 psi. This result was obtained by stress analysis using the finite element method, as shown in Fig. 8. Here, an initial Young's modulus of 300 ksi was assumed for both components, but this value depends on temperature and time due to creep. The 1200 psi value also neglects any softening and subsequent reduction in modulus that may occur due to plasticization by the solvent cement and the subsequent “drying out” of the joint as the solvent dissipates. Thus, while reducing pressure may slow the failure process, any correlation between time to failure and pressure for a particular system that is contaminated with an ESC agent is difficult, if not impossible, to calculate.

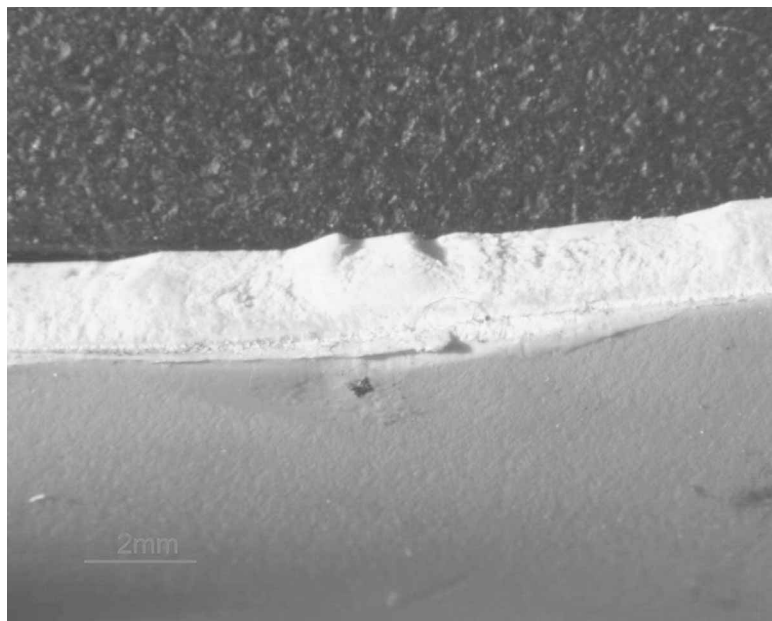


FIG. 9—Fracture origin on a pipe that failed in overstress with evidence of impingement on outer surface.

Regarding maintenance, it is not uncommon for building owners to contract with fire protection companies other than the original system designer and installer for follow-on sprinkler system inspections, tests, and maintenance. Thus, a number of parties may come into contact with the system between the time of installation and any failures. Ascertaining each party's involvement can be critical. For example, companies servicing a fire protection system must ensure that the CPVC sprinkler pipe subject to freezing temperatures is protected using glycerine-based antifreeze. The use of diethylene, ethylene, or propylene glycols is specifically prohibited, as glycol-based antifreeze solutions are known to promote ESC.

Finally, repairs to the sprinkler system or other nearby systems can sometimes introduce inadvertent damage. For instance, there have been failures in PVC lines where a plumber overstressed the pipe by over tightening a connection to an adjacent copper pipe or by bending the pipe to cut in a new metal pipe. Similarly, impact damage caused by repair workers or overstress loads caused by hanging lights or other tools from pipes may introduce cracks that lead to eventual failure. For example, Fig. 9 shows a pipe that failed at the location of conduit bearing on the outside of the pipe. Thus, assessing the repair history of not only the fire protection or plumbing system but also that of other nearby systems may shed light on the failure mode.

Conclusions

While CPVC pipe is a highly competitive product that offers many advantages over metal pipe, it is susceptible to failure by a number of failure modes, including environmental attack. ESC has been observed in a number of installations, primarily in fire protection systems. The presence of a foreign contaminant that can interact with the polymer network is the primary determining factor. Stress is also required, whether resulting from internal pressure, joint connections, hanger restraint stresses, other external sources, or built-in residual stresses. Material defects or variabilities may also contribute to ESC cracking, either by reducing ESCR or providing microscopic stress concentrations. Determining the relative contributions of the contaminant, material condition, and loading factors can be challenging, as a particular ESC agent may not pose a problem under conditions of proper pipe manufacturing and proper installation.

Plumbers, fire protection companies, contractors, building owners, and tenants are encouraged to consult CPVC pipe manufacturers and suppliers for detailed instructions that cover proper storage and handling, joining, assembly, and installation techniques. They are further encouraged to consult the manufacturer's chemical compatibility guidelines, which are readily available and frequently updated on their websites. They should be especially vigilant to monitor the use of fire protection barrier products such as caulks or sealants that may not be compatible with CPVC.

In investigating failures of CPVC and other plastic piping, the forensic engineer has a wide variety of tools at his/her disposal. They include microscopic examination and fractography, mechanical testing, materials analysis, and stress analysis. Utilizing these tools in an efficient and meaningful manner is the key to a successful failure investigation. Still, better tools are needed within the scientific community to assess the relative contributions of stress, environment, and material quality in failures—especially ESC failures. Identifying and quantifying residual stress, thermo-mechanical damage, or other defects in damaged pipe are challenging, and better test methods are needed for forensic investigations.

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Ray L. Hauser¹

Environmental Stress Cracking of Commercial CPVC Pipes

ABSTRACT: A study has been made by the author to learn effects of single and of mixed glycols causing environmental stress cracking of CPVC pipes made by four different manufacturers from two different resin suppliers. The study has shown that a mixture of ethylene glycol and propylene glycol is more damaging to CPVC pipe than either of the glycols by itself in the same relative concentration in water. The study also showed that one pipe resin containing a “tail” of lower molecular weight resin is somewhat more susceptible to cracking than the other resin. A secondary study also shows that the test method is appropriate for quickly evaluating chemicals in anti-corrosion treatments on steel pipes in sprinkler systems that can be particularly damaging to CPVC pipe.

KEYWORDS: fire sprinkler pipe, chlorinated polyvinylchloride, CPVC, environmental stress cracking, glycol, microbially induced corrosion inhibitor

Introduction

CPVC pipe is a good material with many attributes and advantages for fire protection systems. Its most common problem arises from mistakes during installation and maintenance where the wrong anti-freeze chemicals are added to the system or chemicals added to protect steel piping from microbially induced corrosion cause environmental stress cracking (ESC) may defined as cracking caused by tensile stress less than the short-time mechanical strength of a plastic, induced by the chemical environment to which the plastic is exposed.

Glycerol is the only anti-freeze chemical permitted in a fire sprinkler system of CPVC pipe by NFPA Standard 13 [1]. Unfortunately, ethylene glycol, propylene glycol, and sometimes mixtures thereof have been added as anti-freeze

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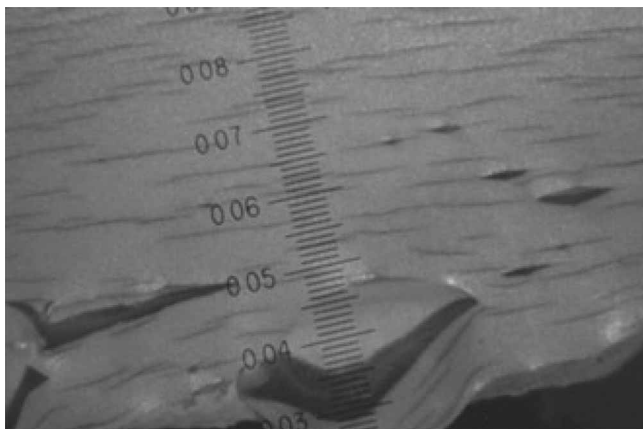


FIG. 1—*Environmental stress cracks from 50 % propylene glycol in a sprinkler system.*

chemicals to some CPVC fire sprinkler systems, with either disastrous results or a threat of disastrous results.

The author has been involved in the examination and failure analysis of a number of installations where glycols have been used, resulting in cracks and bursts such as seen in Fig. 1, or in pre-burst microcracks such as seen in the scanning electron microscope (SEM) photos of Figs. 2 and 3. Figure 4 shows a fine scratch made on the internal surface of a CPVC pipe. Its berm of displaced polymer presents a clear distinction between a scratch and an environmental stress crack. Figures 2–4 and 8 were all taken at the same SEM magnification of $1000\times$.

Goodrich [2] did a major study of environmental stress cracking of CPVC pipe when it was a manufacturer of the CPVC resin. These studies were done

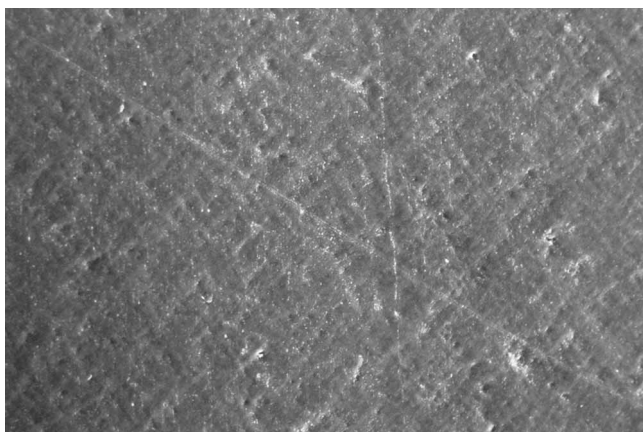


FIG. 2—*V pattern of cracks seen inside of CPVC pipe in fire sprinkler system with mixed glycols, SEM at $1000\times$ magnification.*



FIG. 3—Flaking and crack in CPVC pipe having mixed glycols in sprinkler system.

with the Corzan brand, which is generally a thicker wall than the Blazemaster brand of CPVC resin which is sold by Lubrizol and is used as pipe for fire protection systems at standard dimension ration (SDR) = 13.5.

The first portion of the test program related to mixtures of glycols; the second looked at a chemical that was suspected to cause environmental stress cracks in CPVC pipe. The liquid's molal volume and solubility parameter are factors that affect environmental stress cracking. The molal volumes are 56.0 and 73.5 cm³/gmol, respectively, for ethylene glycol and propylene glycol, and solubility parameters are 32.9 and 30.2 MPa^{0.5} [3].

This accelerated test program (like ASTM standards D1248 and D1693 [4,5]) was intended to provide a means for fairly quick comparison of solvent effects, not to provide design standards like the tests that were done years ago

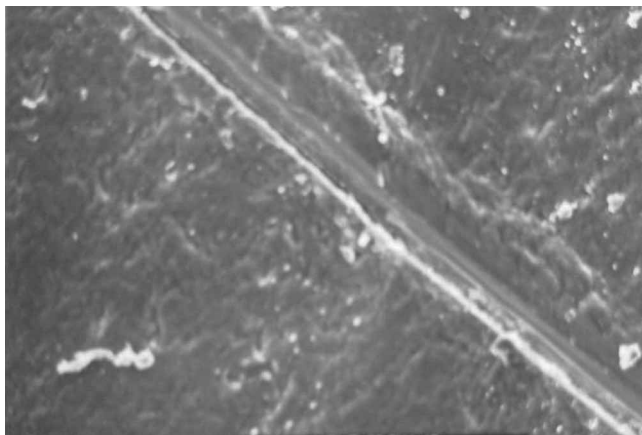


FIG. 4—Scratch made with ophthalmic knife inside of CPVC pipe, showing berms of displaced resin, SEM at 1000 × magnification.

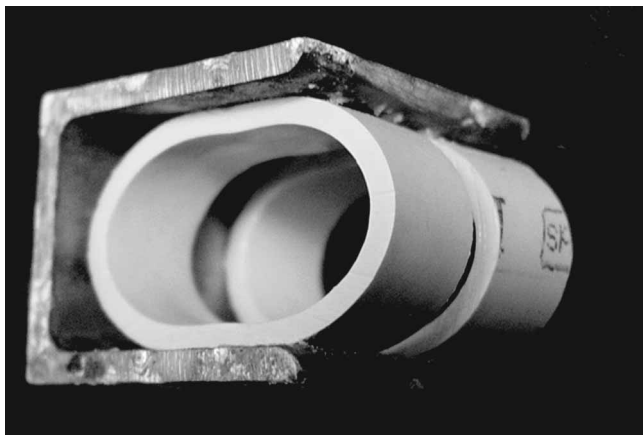


FIG. 5—Rings of CPVC pipe squeezed into aluminum channel for stress testing.

by B. F. Goodrich when CPVC plastics were introduced into the markets for fire suppression systems and for chemical processing. Mixed solvents are not reported in recent editions of these tables, updated by Lubrizol, the present supplier of resins to extruders of Blazemaster pipe.

Mixed Glycols

Having seen no publication or report on the effects of mixed glycols on CPVC pipe, a self-funded study was done, using some rather simplistic means to accelerate the stress and chemical environments. Pipes were obtained from four commercial sources, 1 in. nominal inside diameter with SDR 13.5 rating, from which rings were cut 0.5 in. wide. Clean rings were squeezed in a vise and were pushed into the restraint of aluminum channels having a clearance of 1 in., such as shown in Fig. 5. This test used some of the concepts of ESC test methods ASTM D1693 and D1248, but no notch was placed in the surface for this test program. Thus the nominal 1.31 in. outside diameter pipe was squeezed into an oval flattened to 1 in. outside minor axis.

By using a simplistic calculation of strain = $\Delta(t/2R)$, where t is thickness and R is bend radius), the maximum tensile strain level at the curved outside surface was estimated to be 0.026 and the strain level on the flat inside surface to be 0.050. Both of these strains are well beyond the elastic limit of the CPVC, so the stresses are unknown. These strains are less than those of the two ASTM standards, D1693 and D1248.

The pipe specimens were identified as HB, TB, VB, and VC (commercial sources intentionally not disclosed). These were placed in five different solutions as follows:

1. 50 % ethylene glycol in water (allowable concentration per Goodrich/Lubrizol data for CPVC pipe with Corzan tradename).
2. 75 % ethylene glycol in water (50 % more than the allowable concentration).

3. 25 % propylene glycol in water (allowable concentration per Goodrich/Lubrizol data for CPVC pipe with Corzan tradename).
4. 37.5 % propylene glycol in water (50 % more than the allowable concentration).
5. 25 % ethylene glycol, 12.5 % propylene glycol in water (presumably a mix of the allowable concentrations as equal to a mixture of numbers 1 and 3).
6. 37.5 % ethylene glycol, 18.75 % propylene glycol in water (50 % more than allowable for each).

All of these solutions were prepared with volumetric percentages, which is common practice of the maintenance trade, rather than weight percentages as reported by Goodrich and Lubrizol, but there is very little difference between these two modes.

Jars containing these solutions and the four sets of test specimens were placed in a bath kept at 50°C–54°C, and the specimens were inspected at almost weekly intervals with eyeball examination and often with a 7 × eyepiece. When a crack was seen on the outer surface, it was generally photographed through an optical microscope.

Table 1 presents the sequential observations of cracks in the test specimens.

After 103 or 108 days, one specimen from each pair was removed from the assembly, rinsed with water, measured for springback, and cut into three pieces as shown in Fig. 6, so that both the internal and external surfaces could be examined by optical or scanning electron microscope. Most specimens had cracks visible at about 250 × magnification in an optical microscope, and scanning electron microscope photos at 1000 × magnification showed the presence of different crack and pitting patterns. As seen in Fig. 7, these were not very different from what has been seen in installed sprinkler systems exposed to mixed glycols. Figure 8 shows a mounted and polished cross section of a crack in specimen VC about 25 μm deep.

The springback tests gave an estimate of the residual tensile stress at the outer fiber of distorted specimens from solution 5 as follows in Table 2.

These stresses were estimated by noting the change in $t/2R$ for assumed circular sections at the minimum radii of each ellipse, from the restrained to the relaxed condition, and by assuming 500 000 as the same elastic modulus for each of the pipes. The lesser springback of the 5VC specimen implies more creep and stress relaxation during the test than the other three specimens.

The J sections from tests 2 to 6 were extracted with acetone and the extracts were analyzed by gas chromatograph with flame ionization detector for glycols and these results are shown in Table 3, where 8 mg/kg was the detection limit for these small samples weighing about 2.6 g each. As the VC pipe had been the first to show environmental stress cracking, it also showed greater absorbed glycol content. Whereas propylene glycol appears to be the more potent stress-cracking agent (Table 1), more of the smaller ethylene glycol molecules appear to be absorbed in the CPVC. Ethylene glycol was present at double the concentration of propylene glycol in solution 5.

Tests of molecular weight distribution by gel permeation chromatography of two CPVC resins showed a significant difference that may related to the

TABLE 1—Visual observations of first cracks in outside surface of CPVC pipe.

Jar number and volumetric concentration of ethylene glycol/propylene glycol/water							
Time, days	Bath Temp., °C	1, 50/0/50	2, 75/0/25	3, 0/25/75	4, 0/37.5/62.5	5, 25/12.5/62.5	6, 37.5/18.75/43.75
3	50.4						
10	51.7				VC axial		
17	53.0						
31	50.0						
45	52.4						
59	52.4		VC axial		VC circ TB axial		
66	50.2			VC axial and circ		VC axial	VC axial and circ
73	50.2						
88	53.6	VC					
103 or 108	51.0	HB axial	HB axial	HB, VB	HB, TB, VB	HB, VB, TB	HB, TB, VB

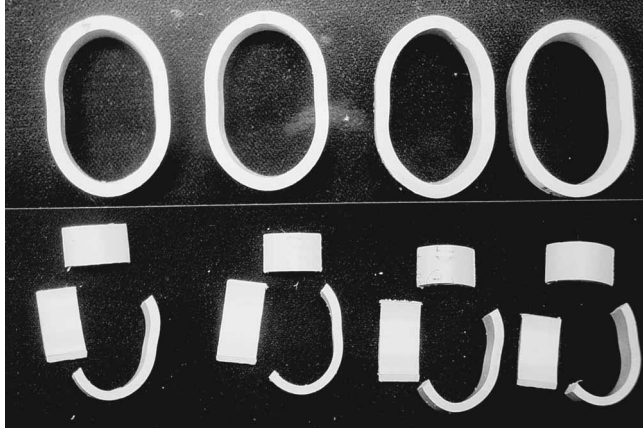


FIG. 6—Cuts of rings after exposure to permit microscopic examination and for extraction of glycols.

differences seen in susceptibility to environmental stress cracking. When samples HB and VC were dissolved in tetrahydrofuran, it was noted that VC dissolved easily at room temperature, but the HB resin required heating to dissolve fully.

Tests of pipe from the two resin suppliers a few years ago showed that the low molecular weight cutoff of one sample was at about 32 min, corresponding to a molecular weight about 1900 Da. The low molecular weight cutoff of the other resin was at about 39 min, corresponding to a molecular weight about 590 Da. However, tests of more recent samples of pipe showed very similar molecular weight distributions of these two resins. There may have been some changes by manufacturers of one of the resins during this interim.

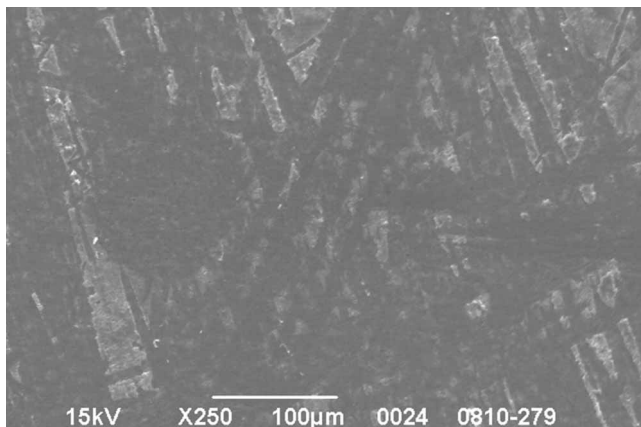


FIG. 7—Interior surface of VB pipe after 108 days in mixed glycols, formula 5.

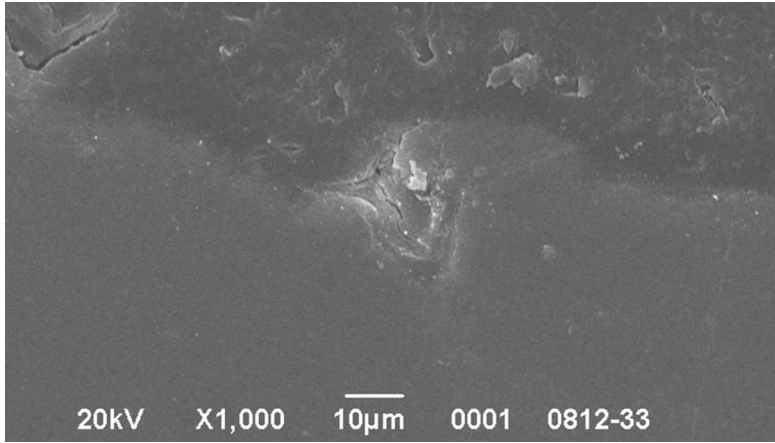


FIG. 8—Cross section of VC pipe showing crack length about 25 μm deep into the pipe wall.

TABLE 2—Estimated residual tensile stresses from solution 5.

Specimen	5HB	5TB	5VC	5VB
Springback, in.	0.047	0.051	0.029	0.048
Residual internal stress, MPa (psi)	13.9	14.8	8.6	13.8
(psi)	(2010)	(2150)	(1250)	(2000)

Other Chemicals

The second experiment was related to a chemical that was the suspected cause (perhaps one of several) of environmental stress cracks in CPVC fire sprinkler systems. The solubility parameter of the suspect chemical was estimated by the method of Snow as reported in the “Hoy Tables of Solubility Parameters,” [6], which was calculated to be $18.0 \text{ MPa}^{0.5}$, dangerously close to the solubility parameter of CPVC (61 % chlorine content) calculated by Goodrich to be $20.1 \text{ MPa}^{0.5}$ [2].

Didecyl dimethyl ammonium chloride was found by chemical analysis (by a resin supplier who assisted with failure analysis) to be present and absorbed in

TABLE 3—Glycol absorption of number 5 specimens.

Sample	Ethylene glycol, mg/kg	Propylene glycol, mg/kg
5HB	13.8	<8
5TB	<8	<8
5VC	115	50.7
5VB	<8	<8



FIG. 9—Cracks seen in CPVC pipe specimen HB after only 42 hours of contact with a quaternary ammonium salt, 250 $\times$ magnification.

CPVC pipe that had been treated with a Microbial Corrosion Inhibitor known to contain quaternary ammonium compounds. This compound was applied (by dip-coating and draining) to rings of pipe samples HB and VC. The alcohol in this Stepan BTC 1010 amine solution was allowed to evaporate. Then the rings were placed within the aluminum channels of Fig. 5. These were placed directly in a water bath at 52°C, and external environmental stress cracks were seen to occur upon the first inspection at 42 hours, as seen in Figs. 9 and 10. Internal cracks could be seen in both specimens with a 7 $\times$ magnifier after 94 hours. This was much faster environmental stress cracking than was seen in any of the

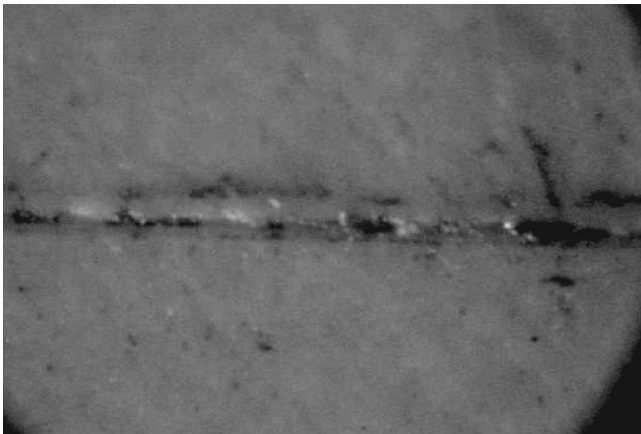


FIG. 10—Cracks seen in CPVC pipe specimen VC After only 42 hours of contact with a quaternary ammonium salt, 250 $\times$ magnification.

glycol tests in the same test fixture and temperature, but it also represented direct chemical contact in a relatively concentrated form.

Conclusions

These very simple test procedures have confirmed that environmental stress cracking can be discerned simply and quickly by optical or scanning electron microscopy, and that the early stage patterns may not be similar to the later fracture patterns. These tests confirm that mixed glycols are more reactive than single glycols, and the tests show that all CPVC resins are not created equal. Although one resin was more susceptible to environmental stress cracking than another brand, the processing variables of extruder machine, rate, temperature and dies are also quality variables, and all commercial CPVC pipe must be considered to be at risk when exposed to the wrong chemicals and mixtures thereof.

These tests also show that the test method is appropriate for testing many chemicals that might come into direct contact with CPVC pipe. A similar alternative that permits timely examination of all surfaces under tensile strain is to place a spacer across the inside diameter of a ring of pipe to form a similar oval.

The VC pipe of this study was withdrawn from production in 2008.

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Amster Howard¹

How to Crawl Through a Pipe—Terminology*

ABSTRACT: Personal inspections, such as “crawl throughs” or “walk-throughs,” of buried pipelines need standardized terminology for describing the interior condition of a pipe. In many cases, personal inspection is less expensive and more effective than a closed circuit television inspection. Remote video inspection of a pipeline interior is a valuable method of inspecting pipeline interiors. However, when accessible, a personal inspection can provide quantitative data not available from video inspection. The Pipeline Assessment and Certification Program (PACP) of the National Association of Sewer Service Companies promotes a uniform system of describing conditions encountered during video inspections of pipe. However, there are no national standards, programs, or instructions for terminology of observations made during a personal inspection. A condition assessment protocol, such as the PACP system, would also be useful. The descriptive terms and nomenclature of the PACP is generally recommended for use in personal pipeline inspections, modified for the means and methods available during “crawl-through” and “walk-through” investigations.

KEYWORDS: pipe, inspection, terminology, acceptance, cracks deformation, CCTV, PACP

Introduction

The interiors of buried pipelines are inspected for many reasons, which include initial acceptance, periodic assessment, investigations of excessive leaks, and failures. Closed circuit television (CCTV) inspection is commonly used for inspecting a pipe interior. New laser profiling methods are also available. For larger pipe sizes, personal inspections provide a more reliable and quantifiable

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method. There are guidelines for the remote inspection methods but not for “crawl-through” assessments.

The National Association of Sewer Service Companies (NASSCO) has adopted and is promoting a Pipeline Assessment and Certification Program (PACP) for companies doing CCTV inspections [1]. This program certifies CCTV operators for assessment of the interior of buried pipe. Certification promotes and unifies descriptive terms and pipeline condition assessments. Pipeline owners and operators should insist that anyone doing a CCTV assessment of their pipeline be PACP certified. An inaccurate and poorly performed video inspection can be seriously misinterpreted. NASSCO has devised a system for describing the interior condition of a pipeline and assessing its status. For the most part, the descriptive terminology is appropriate for evaluation of a buried pipeline. The NASSCO terms are listed in the following discussions. However, when accessible, a personal inspection can provide quantitative data not available from video inspection. This paper gives recommendations for how the PACP terminology should be changed, or varied, for crawl-through inspections. The information presented is summarized from a more comprehensive report [2].

DelDOT Manual

The State of Delaware Dept. of Transportation has developed an excellent manual of procedures and terminology for CCTV inspections of storm sewers [3]. Although based on the NASSCO PACP program, their protocol is more practical, has good illustrations, and includes a procedure for measuring the size of defects. Most importantly, DelDOT requires an accepted joint gap distance be determined for each type of pipe before the inspection so that only *excessive* joint gaps would be noted on the video log. The DelDOT manual is available for downloading from the Internet.

History

In the literature there are references to WRc, MSCCR, MSCC4, NAAPI, PACP, and SPIMA for methods and coding systems for CCTV sewer inspection. The website www.spima.ca has some excellent references that explain these systems and discusses the differences. The site also covers the history of sewer regulation starting in the United Kingdom in 1427. After years of organizations battling between separate systems and methods, NASSCO appears to be the survivor in the United States for a standardized system. However, in Canada, there is a movement toward a separate system as discussed in the Sewer Pipe Inspection and Management Association (SPIMA) website.

PACP Terminology and Suggested Crawl-Through Usage

Note: The PACP terminology for CCTV inspections is presented followed by bold print with the suggested terminology for crawl-through inspections. Where there is no bold print, the PACP terminology is appropriate for crawl-through inspections.

This information is applicable to most types of pipe. However, some sections pertain to a particular type of pipe. Heat fused, welded, or bonded joints are not included in this discussion.

Information for inspection of corrugated metal pipe is given later.

Location Conventions

Numbering—PACP—not used, all longitudinal references are made to the distance the video camera travels.

Crawl Through. Observations should be referenced to each pipe number. The pipe should be numbered looking downstream. However, this is not always convenient or the pipe has been previously numbered with different system. The different numbering convention must be noted and made obvious. On the first pipe, put an arrow under the number “1” to indicate the numbering direction and also put an arrow under the word “flow” to indicate the direction of flow. This marking should be photographed.

The joints should be referred to as J2/3, or the joint between pipes 2 and 3.

Crawl-Through. The pipe number should be marked on the pipe at 3 O’Clock at the upstream end of each pipe section. (*O’Clock is discussed in the following*).

Longitudinal—PACP—Uses 0.1 ft increments going in direction of travel from beginning of camera travel.

Crawl-Through. Typically a spring tape measure with divisions in inches is used. The tang can be hooked on the end of the pipe. The description of how the longitudinal measurement is made should be noted at beginning of inspection notes, e.g., “distances measured from end of pipe barrel at upstream joint.” Or, it may be more convenient to refer the distance to a joint, e.g., 12-3/4 in. from J2/3 in.

Circumferential—PACP—Use O’Clock for circumferential location where 12 O’Clock is top of pipe and 3 O’Clock is springline in direction of travel.

Crawl-Through. Use O’Clock reference looking downstream (not necessarily direction of travel). Useful because subsequent inspections may not be in same direction.

PACP—For observation less than “one hour” in length, use clock reference for center of observation. For observation one hour or more length, use “from” and “to,” *going clockwise*. Use 02 for 2 O’Clock. An observation that goes from 3 O’Clock to 9 O’Clock on the top half of the pipe would be “09 to 03.”

Crawl-Through. For dictating or writing notes while inside a pipe, use “09 to 03oc.” With many numbers possibly being noted, it is useful to add the “oc.”

Cracks, Fractures, and Breaks

The following definitions are paraphrased from the NASSCO PACP Reference Manual, Version 4.1 (NASSCO 2001):

Crack—A crack line is visible on the surface, the crack is not visibly open, and the pieces of pipe are still in place.

Longitudinal crack—A crack that runs approximately along the axis of the pipe. Abbreviate LC. Use clock reference for crack location. If crack path varies, use “wander” as “LC wanders 04 to 08.”

Circumferential crack—A crack that runs approximately at right angles to the axis of the pipe (parallel to the joints). Abbreviate CC.

Fracture—A fracture is a crack that is visibly open although the sections of pipe wall are still in place and not able to move.

Longitudinal fracture—A fracture that runs approximately along the axis of the pipe.

Abbreviate FL

Circumferential fracture—A fracture that runs approximately at right angles to the axis of the pipe (parallel to the joints). Abbreviate FC.

Broken—A pipe is broken where pieces are noticeably displaced and have moved from their original position at least $[1/2]t$ where t = thickness of the pipe wall.

Hole—A hole is where the broken pipe pieces are missing from the pipe wall.

Crawl-Through—The distinction between a crack that is visibly open or not is subjective. A crawl-through has the advantage of being able to measure crack width. In concrete pipe, the crack should be measured using the concrete pipe 0.01-in. crack gage described in ASTM C497 [4]. Cracks that are less than the 0.01 in. width should be designated as “hairline cracks.” Wider cracks in concrete pipe and other pipe can be measured with a feeler gage, spark plug gap wire gage, or a tape measure.

For “broken” description, the pipe thickness may not be known.

Crawl-Through—Hairline crack—crack that is less than 0.01 in. wide for at least four measurements made in 12 in. length. Abbreviate HC.

Crack—Describe width measured four times in 12 in. length. For any crack over 0.1 in. width, describe maximum width.

Longitudinal crack—A crack that runs approximately along the axis of the pipe (parallel to the centerline). Abbreviate LC.

Circumferential crack—A crack that runs approximately at right angles to the axis of the pipe (parallel to the joints). Abbreviate CC.

Crawl-Through—Do not use fracture, see discussion on cracks. Fracture is too subjective.

Broken—A piece of the pipe wall is radially displaced relative to an adjacent piece.

Hole—Broken pipe pieces are missing from the pipe wall.

Deformation

Deformed—PACP—Original cross section of pipe noticeably altered. Estimate vertical and/or horizontal change as percentage of original diameter and state in increments of 5%. These descriptions are basically for flexible pipe.

Crawl-Through. Measure vertical diameters in pipes with oval deformations. If the change in horizontal diameter seems obviously different than change in the vertical diameter, then also measure horizontal diameter. If maximum diameter change is not in the vertical direction, measure and describe as maximum (or minimum) diameter using clock reference. Give deformation as percentage of original diameter, e.g., 3%. In most cases, the nominal diameter of the pipe can be used as the original diameter. However, when the deformation is a critical part of the investigation, the original diameter of the pipe should be determined by averaging several diameter measurements around the circumference perpendicular to the longitudinal axis of the pipe.

Collapse

Collapse—PACP—complete loss of structural integrity of the pipe with more than 40 % of the cross-sectional area lost.

Joints

Offset joint—PACP—An offset joint has the end of one pipe not concentric with the end of the adjoining pipe. An offset joint can be described as the follows:

Medium—the offset is greater than 1.0 pipe wall thickness up to 1.5 pipe wall thickness and

Large—the offset is greater than 1.5 pipe wall thickness.

DelDot describes offset in terms of moderate and severe, where severe is the case where soil or rock is visible. Offset is defined as pipe not being concentric. As wall thickness is difficult to ascertain from video, the DelDOT description is more useful.

Crawl-Through—The offset of a joint between adjacent pipe should be measured with a carpenters square or tape measure. Measure the largest offset and note the O'Clock position. Abbreviate JO and measure the downstream pipe relative to the upstream pipe. For example, J34/35 JO 3/4 in ↓ 6oc, which means at the joint between pipes 34 and 35, the downstream pipe is 3/4 in. lower than the upstream pipe, measured at the invert.

Separated (open) Joint—PACP—A separated joint has a visible gap between the ends of adjacent pipe. (Earlier terminology used “open joint” but the latest version of PACP recommends calling it a separated joint and uses the abbreviated “JS”.) PACP describes a separated joint as follows:

Medium—the separation is greater than 1.0 pipe wall thickness up to 1.5 pipe wall thickness and

Large—the separation is greater than 1.5 pipe wall thickness.

DO NOT USE THE PACP LANGUAGE. Most pipe will have a gap between the bell and the spigot end. A gap is desirable so the joint can move as the ground shifts. If there is no gap and one pipe moves at an angle to the pipe centerline, one side of the joint gets crushed. “Separated” or “open” sounds like a defect. There can be a separation and the seal provided by the gasket is still intact. The term “open” should only refer to a joint where there is evidence of infiltration or exfiltration, or the possibility of either.

DelDOT uses “open” and describes the condition as moderate or severe where severe means that soil or rock is visible. However DelDOT requires that the video operator calibrate the video equipment so that distances can be measured. The calibration involves placing a ruler against the pipe wall at 3, 6, 9, and 12 O’Clock and measuring image on video screen. Then DelDOT has tables of acceptable gaps between pipe types that can be used to see if gap is excessive. However, they still refer to any gap as a “minor defect.”

***Crawl-Through*—The space between the ends of the pipe should be referred to as a “gap” and the gap should be measured if the gap seems excessive. Before entering the pipe for inspection, the allowable gap between the ends should be determined from manufacturer or manufacturer’s literature, as required by DelDOT. If the pipe was pulled to make a vertical or horizontal curve, the allowable gap should also be ascertained.**

The joint gap should be abbreviated JG. For example, J34/35 JG 2-1/2 in 3oc, JG [1/4] in 9oc, which means at the joint between pipes 34 and 35 there is a gap of 2-1/2 in. es at the 3 O’Clock position and a gap of 1/4 in. at the 9 O’Clock position.

Infiltration

***Infiltration*—PACP—Infiltration is the ingress of groundwater into the pipe. Infiltration is described as follows:**

- Weeper—no visible drips,
- Dripper—water dripping,
- Runner—water continuously flowing,
- Gusher—water entering pipe as if under pressure.

Surface Damage

For concrete pipe, PACP has eight levels of surface damage, as follows:

- 1—roughness increased,
- 2—aggregate visible,
- 3—aggregate projecting,
- 4—aggregate missing,
- 5—reinforcement visible,
- 6—reinforcement projecting,
- 7—reinforcement corroded,
- 8—missing pipe wall.

Concrete spalling, metal corrosion, detached lining, missing lining, blisters, and buckled are also terms used by PACP.

Culvert Inspection

The Federal Highway Administration has a "Culvert Inspection Manual" that contains descriptive terms for metal and concrete culverts [5]. These terms are also referred to in the "Culvert Repair Practices Manual" [6], and should be considered the standard for describing conditions and defects in corrugated metal pipe. Terms such as bolt tipping, cocked seams, and cusped seams are explained and illustrated. The NCSPA Design Manual has a chapter on "Inspection, Maintenance, and Rehabilitation" and several references on inspection [7].

Summary and Conclusions

The National Association of Sewer Service Companies (NASSCO) has adopted and is promoting a Pipeline Assessment and Certification Program (PACP) for companies doing CCTV inspections [1]. NASSCO has devised a system for describing the interior condition of a pipeline and assessing its status from viewing videotape. Some of the PACP terminology should be modified for "crawl-through" or "walk-through" inspections where defects or conditions can be directly observed and measured. Although some suggested modifications are minor and related to direct observation, the descriptive PACP terms for joint gaps should be replaced with more appropriate language.

Inspection manuals from the Federal Highway Administration should be used to describe defects in corrugated metal culverts.

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NEW MATERIALS AND APPLICATIONS

Frank Volgstadt¹

Yesterday, Today, and Now: Polyamide-11 Gas Piping at 200 psig Under the New Rules

ABSTRACT: This paper will explore and explain the Dec. 24, 2008, PHMSA rulemaking resulting in revisions to 192.123, permitting the use of polyamide-11 (PA-11) plastic gas piping in gas distribution systems at pressures of up to 200 psig using a design factor of 0.4 without waivers or special permits. The paper will also explore (1) PA-11 properties and attributes; (2) PA-11 and polyethylene piping in complimentary but not competing plastic gas piping applications; (3) PA-11 piping as a cost effective alternate to steel piping; (4) updates on some existing, special permit (waiver) installations made prior to the Dec. 24, 2008, rulemaking; (5) standard PA-11 installations as well as unique applications into which PA-11 piping is being introduced; and (6) how to install PA-11 piping.

KEYWORDS: polyamide-11, PA-11, gas pipe, plastic pipe, high pressure, gas distribution, nylon-11

Introduction

Late in December 2008, the Department of Transportation (DOT) issued a Final Rule to amend U.S. DOT Code of Federal Regulations (CFR) Title 49, Part 192, to permit the use of polyamide-11 (PA-11) gas distribution piping to be installed and operated at pressures up to and including 200 psi in a design factor of 0.40. This is the new rule described in the title of this paper. This paper will describe the arduous efforts by many parties to provide sufficient technical and practical information studies that are involved in the development and introduction of a new, safe, and reliable high pressure thermoplastic gas piping material. Obtaining the Final Rule was not an easy task, but it is a necessary task, taking some 17 years from the first U.S. commercial introduction of PA-11 in the manufacture of mechanical fittings and some 13 years following the addition of PA-11 as

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Annex A5 to ASTM D2513. We finally made it; polyamide-11 gas piping is now permitted to be installed in jurisdictional distribution piping at pressures up to and including 200 psi using a design factor of 0.40 without the need for special permits or waivers.

Background

PA-11 is a high performance “nylon” thermoplastic material that was first developed in the 1940s. It became a commercial plastic in the 1950s. The raw material for the production of the PA-11 monomer is castor beans; thus, it is one of the few thermoplastics that are not derived from petroleum stocks. It is truly a “green” plastic manufactured from castor beans, a renewable resource. PA-11 has been produced in the United States since 1971 and in France since the 1950s. PA-11 gas piping is now a fully approved piping material for installations throughout the United States in SDR 11 sizes up to and including 4 NPS at pressures up to 200 psig using a design factor of 0.40 under U.S. DOT CFR Title 49, Part 192. Approvals and standards are fully in place for PA-11 piping, butt fusion fittings, electrofusion fittings, gas valves, mechanical fittings, and auxiliary appurtenances. In addition to ASTM D2513, a model specification is available, as well as a standardized, proven, and recommended butt fusion procedure. The various codes and standards are detailed later in this paper as they include numerous standards and code organizations such as ASTM, NFPA 58, NFPA 54, ANSI/ASME B16.40, ASME B31.8, ANSI Z380, and ISO DIS Standards.

Complete PA-11 gas piping systems are available from George Fischer Central Plastics, and installations are permitted under U.S. CFR Title 49, Part 192, without a waiver, at pressures up to and including 200 psig using a 0.40 design factor.

Early History of the Use of Polyamide-11 in Gas Distribution Applications

The first use of PA-11, under the trade name RILSAN[®], was in Australia. It was used in the early 1970s to rehabilitate city manufactured gas distribution piping by insertion renewal. The piping used was in SDRs 25–33, which is very thin-wall as compared to today’s polyethylene (PE) gas distribution piping, which is typically SDR 11 in sizes 4 NPS and smaller. Even considering early PA-11 pipings’ relatively thin walls in SDRs 25–33, the piping was still rated at pressures up to and including 30 psig. A total of 13 000 miles of PA-11 gas piping was installed in Australia. These early PA-11 Australian piping systems were joined by solvent cemented socket type couplings. Today’s modern PA-11 gas piping systems are joined by heat fusion, butt fusion, and electrofusion, in a similar manner to fusion joining of PE gas piping.

Potential Applications of Polyamide-11 Gas Piping

- Replacement renewal of steel pipe at pressures up to 200 psig with PA-11.

- New high pressure, up to 200 psig, PA-11 line extensions and feeder lines to subdivisions and outlying communities. Plowing in extended lengths of coiled PA-11 piping with minimal joints and minimal disruption of the surrounding area is a very effective method of providing gas service to outlying communities.
- Bridge crossings where higher temperatures are anticipated based on PA-11 being the only commercial thermoplastic gas piping system having a PPI TR4 180°F (82°C) hydrostatic design basis (HDB).
- Contaminated soil (liquid hydrocarbon), “brown fields,” where PA-11 can be plowed in with minimal disturbance of the soil. This is based on PA-11’s chemical derating factor of 1.0 in these types of installation environments. PE gas piping has a chemical derating factor of 0.5 for these applications.
- Horizontal directional drilling based on PA-11’s excellent abrasion resistance and superior slow crack growth resistance. In addition, it has been demonstrated that PA-11 HDD installations under waterways are a very cost effective method as compared to steel piping because of the flexibility of PA-11 piping as compared to steel piping, thus requiring much shorter, steeper lead ins.
- Pipe bursting without the need for a protective sleeve based on the abrasion resistance and superior slow crack growth resistance of PA-11 piping.

Cost Savings

Utilities and local distribution companies are finding that the PA-11 is a very cost effective alternative to steel piping based on installed costs, as detailed below. This does not even take into consideration the cost of installing and operating the pipeline over its lifetime. This is detailed below to take into consideration the additional costs of the cathodic protection and monitoring of coated steel. Cost numbers and comments were captured in the tow waiver installations below.

Public Service of New Mexico

Public Service of New Mexico (PNM) installed 2.5 miles (13 000 ft) of 2 NPS PA-11 pipe as an alternative to installing 2 NPS coated steel pipe. The total installed cost savings for using the PA-11 piping was about \$200 000 in their lifetime savings; no cathodic protection is estimated at greater than \$500 000.

Arkansas Western Gas

In 2007, Arkansas West Gas (AWG) installed 7900 ft of 4 NPS SDR 11 PA-11 piping. This installation was the first ever coiled 4 NPS PA-11 installation, and the first PA-11 directional bore under a river. The system is currently operating at 190 psig. Although actual state savings figures in dollars were not made available, the following comments were a result of the PA-11 installation of the 4 NPS pipe.

River Crossing Bore—

“The flexibility of the plastic pipe [PA-11], as opposed to steel, eliminated the trajectory issue. With the 4 in. PA-11 pipe, AWG started closer to the river and reached the necessary depth with less pipe – cost savings number 1.”

“While steel is bendable, extra fittings must be installed and larger equipment is needed to make the bend and lower the steel and the place. The makeup of plastic [PA-11] allows for larger directional bends without the need for extra equipment—cost savings number 2.”

Joints—

“Perhaps the largest cost savings from PA-11 piping came in the time saved on fusions. AWG estimates it saved 30 to 40 min per joint using the PA-11 pipe. Creating a butt fusion joint with PA-11 pipe takes no more than 15 min to complete.”

Chronology of the Use of Polyamide-11 Gas Piping in the United States

The first commercial use of PA-11 was in 1992 in the molding of mechanical fittings for use in the connection of PE gas distribution piping complying with ASTM D2513. This application successfully continues even today. First discussions with the U.S. DOT regarding the use of PA-11 as an approved gas piping plastic under ASTM D2513 were started in 1995. Based on these discussions, comprehensive research and development studies were initiated with the Gas Research Institute (GRI) in Chicago, IL. The objective of these studies was to exhaustively evaluate PA-11 as a safe and effective gas piping material for use in gas distribution systems throughout the United States and Canada. The performance requirements contained in ASTM D2513 were used as the template for these evaluations. Also included in the GRI studies were extensive studies on squeeze-off, joining procedures including butt fusion (see Fig. 1) using the same equipment as that used in butt fusion of PE and short-term and long-term properties. Conclusions, in part, are as follows:

“PA-11 has met or exceeded all of the provisions contained within ASTM D2513-99 Appendix X1, *NEW MATERIALS*, for the use of new materials and underground natural gas distribution application.”

“PA-11 can be successfully installed and operated using conventional construction and maintenance practices specific to PE. Comments received from the construction crews did not indicate any special considerations were required when installing PA-11 piping systems as compared to PE.”

“Overall, the results of the comprehensive short-term and long-term testing described in this report indicate that the PA-11 pipe is a suitable plastic alternative to steel systems operating at higher pressures and under exposure to high temperatures for a short period of time.”



FIG. 1—High pressure PA-11 piping ready for installation.

The detailed results of this extensive GRI study are contained in three GRI issued reports issued as a follows:

- *Final Report—Part I Technical Reference on the Physical, Mechanical, and Chemical Properties of PA11 Pipe Materials for Use in Gas Distribution Systems Operated at Higher Pressures and Temperatures*—David D'Ambrosio, John Wilson—December 1998
- *Final Report—Part II Evaluation of PA11 Piping for Use in Gas Distribution Systems Operating at High Pressures and Temperatures* —Hitesh Patadia, Greg Konwinski—February 2000
- *Final Report—Part III Field Installation and Aging Characteristic Evaluation of PA11 Piping for Use in Gas Distribution Systems Operating at High Pressures and Temperatures*—Hitesh Patadia, Duc Le —May 2000

As part of the ongoing work being done by the GRI, ASTM D2513 was revised to add PA-11 as an approved gas piping material for use in pipes and fittings. This was first published as ASTM D2513-96. The first PA-11 gas piping requirements are contained in ASTM D2513-96a, mandatory Annex A5.

In 1997, the first trial installation of PA-11 gas piping was successfully accomplished on Nicor Gas property.

As part of ongoing discussions with the U.S. DOT Public Right of Way (PROW), installations were requested to demonstrate the piping, installation techniques, and joining techniques. The first of six PROW installations were installed in 1999.

By 2004, all necessary codes and standards revisions for the inclusion of PA-11 gas piping were completed, and a full 2 NPS piping system was available for additional waiver/special permit installations.

Finally, in 1995, a petition was submitted to the U.S. DOT to amend CFR Title 49, Part 192, to permit the use of PA-11 gas piping in sizes up to and

including 4 NPS, complying with ASTM D2513, at pressures up to and including 200 psig using a design factor of 0.40. The petition was granted and a Final Rule was issued, some 13 years later, in December 2008, to permit the use and installation of PA-11 gas piping systems. As part of this rulemaking, 192.121 and 192.123 were amended and the 192.7 reference standard is ASTM D2513-99.

As of 2009, there are activities taking place within ASTM International to make ASTM D2513 a PE only standard. These actions, should they be successful, would mean that all materials other than PE would be removed from the current edition of ASTM D2513. This will not affect the current or future use of PA-11 piping under Part 192, since the PA-11 containing 1999 edition of ASTM D2513 will continue to be referenced in the U.S. Federal Code of Regulations. There is currently a project within ASTM subcommittee F17.60 to write a stand-alone PA-11 gas piping standard which will become referenced in Part 192 of the CFR once it has been published.

The following is a list of codes, standards, and reference documents which currently apply to PA-11 piping and form a complete complement of support standards for complete PA-11 piping systems.

ASTM Pipe and Fitting Standards

- ASTM² D2513, an American national standard, Standard Specification for Thermoplastic Gas Pressure Pipe, Tubing, and Fittings
- ASTM F1733, an American national standard, Standard Specification for Butt Heat Fusion Polyamide (PA) Plastic Fitting for Polyamide (PA) Plastic Pipe and Tubing
- ASTM F1973, an American national standard, Standard Specification for Factory Assembled Anodeless are Risers and Transition Fittings in Polyethylene (PE) and Polyamide 11 (PA11) Fuel Gas Distribution Systems
- ASTM F2145, an American national standard, Standard Specification for Polyamide 11 (PA 11) Mechanical Fittings for Use on Outside Diameter Controlled Polyamide 11 Pipe and Tubing
- ASTM F2600-06, an American national standard, Standard Specification for Electrofusion Type Polyamide-11 Fittings for Outside Diameter Controlled Polyamide-11 Pipe and Tubing

Valves

- ASME³ B16.40, an American national standard, Manually Operated Thermoplastic Gas Shutoffs and Valves in Gas Distribution Systems

Piping Codes

- U.S. DOT Code of Federal Regulations (CFR) Title 49, Part 192

²ASTM International, 100 Barr Harbor Drive, P.O. Box C700, West Conshohocken, PA 19428-2959.

³The American Society of Mechanical Engineers, Three Park Avenue, New York 10016.

- CSA-CAN B137.12 Polyamide Piping Systems for Gas Services
- ASME B31.8, an American national standard, Gas Transmission and Distribution Piping Systems, ASME Code for Pressure Piping, B31
- NFPA⁴ 58, an American national standard, Liquefied Petroleum Gas Code
- NFPA 54/ANSI Z22 3.1, National Fuel Gas Code

Other Industry Standards

- GPTC⁵ Guide/ANSI Z380, an American national standard, Guide for Gas Transmission and Distribution Piping Systems
- PPI⁶ MS 4 Model Specification for Polyamide 11 (PA-11) Plastic Pipe, Tubing and Fittings for Fuel Gas Distribution Systems
- PPI TR-3 Policies and Procedures for Developing Hydrostatic Design Basis (HDB), Strength Design Basis (SDB), Pressure Design Basis (PDB) and Minimum Required Strength (MRS) Ratings for Thermoplastic Piping Materials or Pipe
- PPI TR4 PPI Listing of Hydrostatic Design Basis (HDB), Strength Design Basis (SDB), Pressure Design Basis (PDB) and Minimum Required Strength (MRS) Ratings for Thermoplastic Piping Materials or Pipe
- PPI TR-45 Butt Fusion Joining Procedure for Field Joining of Polyamide-11 (PA-11) Pipe

Key Polyamide-11 Piping Properties

Superior Resistance to Slow Crack Growth

This is demonstrated by the following test results on polyamide-11

- No slow crack growth failures in 176 °F (80 °C) long-term hydrostatic stress rupture testing after 8000 h
- No slow crack gross initiation in PENT testing at 176 °F (80 °C) and 350 psi (2.4 MPa) stress after 2000 h
- No slow crack growth after 4000 h when tested in the Gas Technology Institute (GTI) three-point band sector testing at 158 °F (70 °C)/17 lb (7.7 kg)

Superior Resistance to RCP

- Full-scale ISO 13478 RCP testing on 4 NPS SDR 11 PA-11 piping resulted in a critical pressure of 435 psig. That is, RCP is not a limiting factor for PA-11 piping in sizes up to and including 4 NPS

⁴National Fire Protection Association, Quincy, MA 02169.

⁵Gas Piping Technology Committee, Secretariat AGA, Washington, D.C., www.aga.org/pubs.

⁶The Plastics Pipe Institute, Inc., www.plasticpipe.org.

TABLE 1—HDB values from PPI TR4.

SDR 11	73°F	140°F	180°F
Yellow PA-11 (psi)	2500	1600	1250
Black PA-11 (psi)	3150	2000	1600
PE3708 (psi)	1250	1000	Not rated
PE4710 (psi)	1600	800	Not rated

Superior Pressure Ratings and Elevated Temperature Performance

Hydrostatic Design Basis—

Although PA-11 is not intended as a replacement piping to PE, the below comparisons gives the readers some insight to the increased pressure ratings and design pressures available to the users when using PA-11 piping as an alternative to steel piping. Per the below Table 1, yellow PA-11 piping has 1–1/2 to 2 times the 73°F (23°C) HDB of PE piping and black PA-11 piping has 2–2.5 times the HDB as listed in PPI TR4. Similar differences in the HDB values are noted at 140°F (60°F) and PE is not even rated at 180°F (82°C). Factoring in the difference in design factors as permitted under Part 192, Table 2 illustrates the differences in design pressures calculated for the different materials. Note that in Regulated installations in the United States, the design pressure is limited under 192.123 to 125 psig for PE and 200 psi for PA-11.

Waiver/Special Permit Polyamide-11 Field Test Installations

As part of the introduction of new plastic plating systems into the gas distribution market, an extensive field test program must be undertaken in order to prove the piping system and materials, and more importantly, in order to weed out any potential installation problems. This was the objective of the PA-11 waiver installations. In addition, the DOT requested that the waiver installations be conducted in a variety of installation environment extremes. PROW installations were a must. Thus, the following environmental extremists were selected:

- Wet environment.
- Rocky environment.
- Cold environment.
- Hot/dry environment.

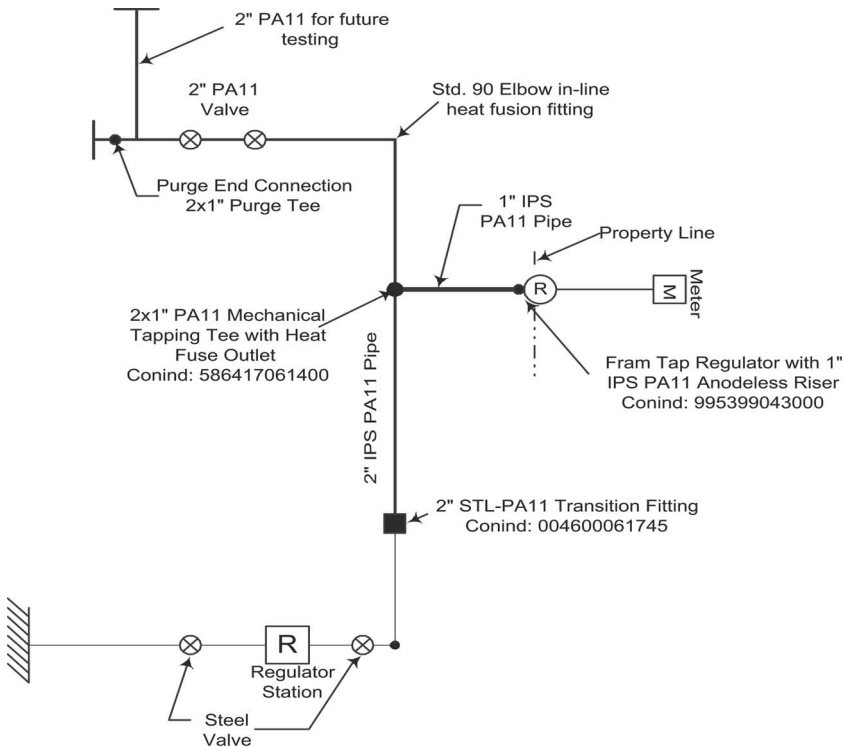
TABLE 2—Calculated design pressures for PA-11 and PE piping.

SDR 11	73°F	140°F	180°F
Black PA-11 _{0.40} (psig)	252	160	128
Yellow PA-11 _{0.40} (psig)	200	128	100
PE3708 _{0.32} (psig)	75	64	NA
PE4710 _{0.32} (psig)	102	64	NA

Table 3 outlines the PA-11 installation locations, operators, installation environments, pressures, sizes, completion dates, and comments.

Details of Some of the Waiver Field Test Installations

City of Mesa Installation—As part of a series of trial installations requested by the DOT, the Gas Technology Institute (GTI) arranged an installation in the City of Mesa's distribution system in Queens Creek, AZ. The below figure presents a schematic of the installation site.



At the onset of the installation, butt fusion problems were experienced when fusing 40 ft sticks of RDG 179 based pipe. The installation operators noted that the appearance of the fusion beads was significantly different than those produced during a training and qualification exercise that took place in April 2002 at the City of Mesa's facilities. Additionally, it was noted that some of the pipe exhibited a brittle failure when impacted.

The installation was halted and an investigation into the cause of the fusion problems and brittle pipe failures was initiated by the GTI and Atofina (now Arkema) Chemicals, Inc. The results of that extensive study determined that the problems were due to changes in the yellow pigment that was recently changed in the PA-11 formulation. The new pigment caused unanticipated premature oxidation of the pipe surface. The formulation was immediately returned to the original yellow pigment. There is no evidence whatsoever that there are any

TABLE 3—Summary of waiver installations of PA-11 piping from 1997 through 2004.

Utility	Motivation	Pressure/Length	Comments	Status
Nicor Gas Naperville, IL	First trial	150 psig/500 ft	Private property	Complete 10/1997
Nicor Gas Woodstock, IL	First PROW installation	150 psig/1 mile	No hot tapping	Complete 12/1999
Atmos Energy Choctaux, LA	"Wet" environment	90 psig/500 ft	No waiver required	Complete 2/2002
Piedmont Gas Nashville, TN	"Rocky" soil conditions	175 psig/6000 ft	Use 0.4 DF Service tees installed at line pressure	Complete 4/2003
Questar Delta, UT	"Cold" climate	200 psig/4000 ft	Use 0.4 DF maximum MAOP for SDR 11 pipe	Complete 6/2003
City of Mesa Mesa, AZ	"Hot/dry" environment	102 psig/800 ft	Using HDB at 140°F, DF = 0.32 installation depth = 5 ft. staged P increase	Complete 6/2004
PNM Lake Arthur, NM	"Hot/dry"	175 psig/2.5 miles	Use 0.4 DF 1000 ft coils	Complete 11/2004
Arkansas Western Gas, Fayetteville, AR	Soft soil and river crossing	195–200 psig	195–200 psig first installation of 4 NPS PA-11	Completed 2007



FIG. 2—Coiled PA-11 pipe after straightening, ready to be rolled into the trench.

oxidative problems in pipes or fittings made from the original yellow or black PA-11 formulations for gas piping applications.

The bottomline is that, to paraphrase comments from Richard Sanders of the DOT, this is the reason we require extensive full-scale field test installations; to weed out these kinds of problems before full commercial introduction of a new plastic piping material occurs. The system works.

Public Service of New Mexico Installation

Near Lake Arthur, NM, starting on November 8 and continuing until November 12, PNM installed just under 13 000 ft of Rilsan[®] RDG-179/R gas pipe fabricated by George Fischer Sloane.



FIG. 3—Loading 1000 ft coils of PA-11 Pipe onto installation trailer.

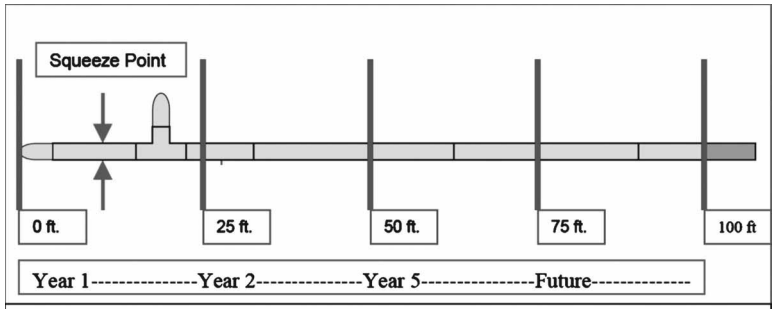


FIG. 4—Layout of PA-11 coupon removal test leg for PNM installation.

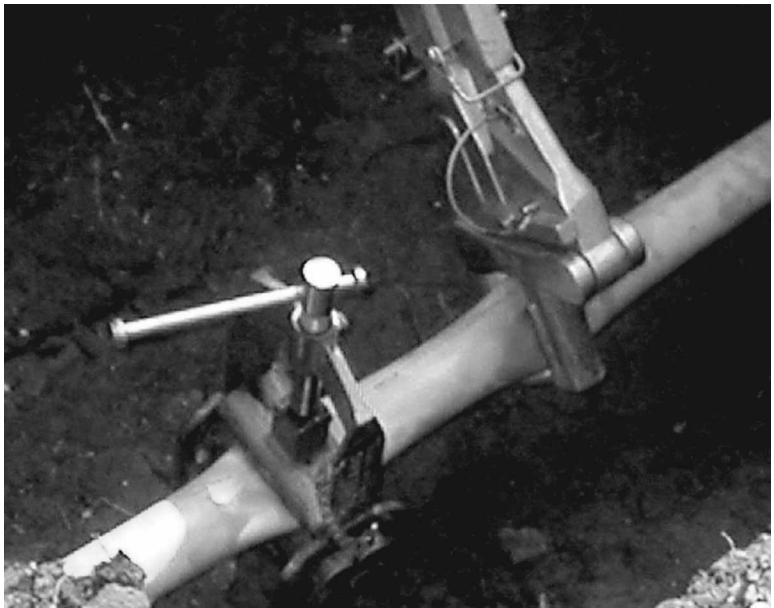


FIG. 5—Two squeeze-off methods on PA-11.



FIG. 6—Typical butt fusion joint in 2 NPS PA-11 piping.

The pipe was delivered to PNM's Roswell, NM, facility about two weeks in advance of the installation and was stored outdoors without special covering. There were seven coils of nominally 1000 ft and 12 coils of nominally 500 ft (Fig. 3).

The new PA-11 pipe was to replace an aging and leaking 2 in. SDR20 + ABS pipe operating at 10 psig. The stick ABS pipe was joined at 40 ft intervals with adhesive socket couplings. There is a single farm customer on the pipeline with two metered services, making the 2 in. high pressure line significantly over-designed, but the purpose of the installation is demonstration of construction and operating techniques for future use in PNM after DOT approval.

The new installation included a 100 ft long dead leg to be used for 25 ft long sample removals at 1, 2, 5, and more-than-five year removals. (Fig. 4) The 1 year section contained a pipe-pipe butt fusion, a squeeze-off point, and a butt fusion tee with one leg terminated by a butt fusion cap. The remaining segments had pipe-pipe butt fusions (Fig. 5). The sample recovery cut points and various segments and features of interest were clearly marked with an indelible pen (Figs 6–11).

It took almost half of Tuesday to construct the dead leg because of the numerous butt fusions. In the remaining 4 h of daylight in the afternoon, we installed 3500 ft of pipe using three each of 500 ft and two each of 1000 ft coils. On Wednesday, we installed 5500 ft using five each of 1000 ft coils and one 500 ft coil. We would have installed more, but we spent several hours waiting for a new trenching machine to arrive from El Paso to replace the existing one that had broken down late on Tuesday. On Thursday, the pace was much slower due to the need to pass under a large steel cattle gate and cross a road. The PNM crew also had to locate and safely cross a buried telephone line. Both of



FIG. 7—Butt fusion of PA-11 piping using standard PE butt fusion equipment.

these complications required lots of hand digging and careful backhoe work. We were able to install only about 2500 ft. The job was finished on Friday using 850 ft of pipe from a 1000 ft coil. 2 in. $\times$ 1 in. tapping tees (Fig. 12), a 1 in. SDR-11 pipe, and risers were used to tie in the regulators and meters on the two services. The new system was put on 320 psig of natural gas as a pressure test for 15 h. On Saturday morning, the pressure test was complete and PNM reduced the pressure to 175 psig. They opened the valves to the meters and started delivering gas to the customer.

The new trencher made as much as 15 ft/min (900 ft/h) of ditch, which was an improvement over the old one, but we were able to load and lay down 1000 ft of pipe and do the fusion in less than 60 min.

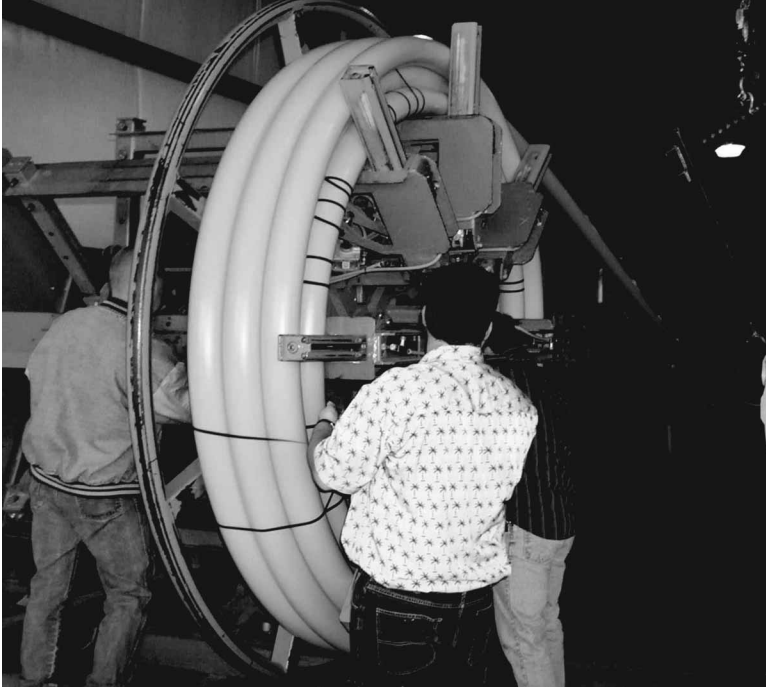


FIG. 8—Production of 4 NPS PA-11 gas piping.

Nashville Gas Polyamide-11 Installation⁷

The installation is in a class 3 location serving eight customers to date. ~5723 ft of 2 in. SDR1 1 PA-11 main was installed using the open cut technique. ~2437 ft of 1 in. IPS SDR1 1 pipe was used to serve the customers up the house meter set.

A 2 in. valve tee was installed to the 8 in. steel line, which operates at 175 psig. The 2 in. steel line was extended below NW Rutland Rd. One of the conditions throughout all of the installations is the removal of pipe sections at periodic intervals to evaluate the effects of in-service conditions. A test leg was constructed using a 1 in. valve tee on top of the 2 in. steel main prior to crossing Rutland Rd. A 2 × 1 in. steel reducer coupling was installed and a small piece of 2 in. steel was installed. A 2 in. steel to PA-11 transition fitting was installed to facilitate the transition to PA-11. ~10 ft of PA-11 was installed and connected to a three-way PA-11 tee, an additional 10 ft of PA-11 pipe was installed in both directions from the three-way tee to serve as the test pipe after 12 and 24 months of service. A 2 in. bolt-on mechanical tee was installed to serve as the purge point for the test loop.

⁷Field Installation and Aging Characteristic Evaluation of PA-11 Piping Systems at Elevated Pressures—Nashville Gas, Prepared for: Arkema, Inc., Prepared by: Dennis R. Jarnecke.



FIG. 9—*Trial production of 8 NPS PA-11 gas piping.*

After the road crossing, a 2 in. valve was installed in order to isolate the test loop and a transition fitting was installed to extend the project using PA-11 piping. ~ 4300 ft of PA-11 pipe (40 ft sticks) was installed following the curvature of Rutland Rd. It is important to note that no PA-11 elbows (90°) were used in this installation. The entire PA-11 pipe was subjected to the bend radius of the street crossings.

2 × 1 in.² mechanical tees with a heat fuse outlet were installed on to the PA-11 main and a 1 in. PA-11 pipe was fused from the outlet of the tee to the house meter sets operating at 175 psig.

Following the turn towards Mt. Juliet Rd., an additional 1000 ft of 40 ft stick pieces of PA-11 pipe were heat fused and installed. An additional 500 ft of coiled 2 in. PA-11 was installed for the final stretch. An inverted 2 in. steel-PA-11 transition fitting was installed along with appropriate end connections to facilitate the purge.

Once installed, the PA-11 piping system was subjected to a pressure test. One on the main points of the waiver was to pressure test the installation at two times the operating pressures versus the 1.5 times required by the code. As a result, the entire line was pressurized to 350 psig and allowed to remain under test for 72 h. The pressure was reduced and the line was purged using conventional purging practices and the installation was live on April 23, 2003,

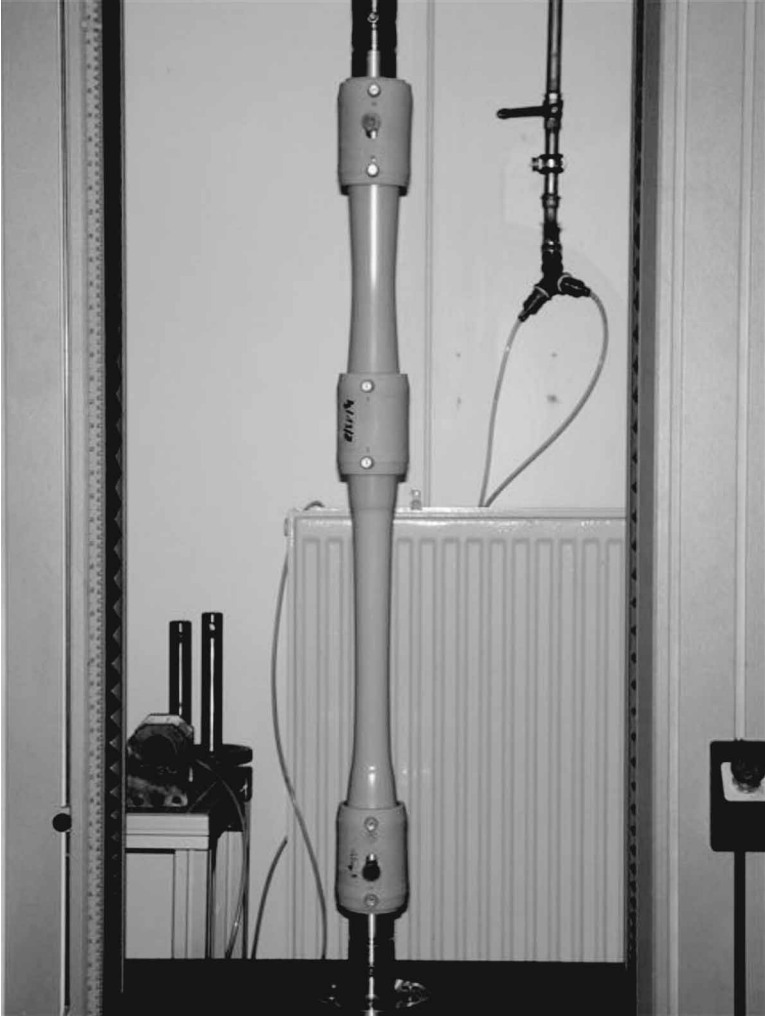


FIG. 10—*Tensile testing of 4 NPS PA-11 electrofusion couplings.*

operating at 175 psig with three service lines on that day. Additional service lines have been installed since.

A PLCS coiled pipe trailer and a 2 in. pipe straightener were used to uncoil the coiled PA-11 pipe. One 500 ft coil of 2 in. PA-11 was installed on the trailer. A backhoe was used to pull the pipe off of the trailer and through the straightener. The use of the coiled pipe greatly increased the speed at which the pipe was installed. Butt fusions are only required every 500 ft instead of every 40 ft as with the stick pipe.



FIG. 11—Using mechanical couplings and tapping tees on PA-11 gas piping.



FIG. 12—No puncture of backhoe tooth. Note entire backhoe, wheels and all, off the ground.



FIG. 13—*Close-up of backhoe tooth impingement.*



FIG. 14—*No puncture.*

Arkansas Western Gas

Arkansas Western installed 7900 ft of 4 NPS SDR 11 PA-11 piping. An article in the Pipeline and Gas Journal October 2008 details that 2007 installation, which included a river crossing by directional bore. The installation was very successful in that section is currently being operated at 195 psig. Figures 13 and 14 below are photographs of attempts to puncture the 4 NPS PA-11 piping at AWG with a backhoe tooth and are very revealing, as no puncture occurred.

Future Planned Polyamide-11 Installations Under the New Final Rule

Atmos Energy—Atmos Energy is in the final stages of preparation for the installation of 6000 ft of 4 NPS SDR 11 PA-11 piping. The word is that if this installation is successful, as it is expected to be, Atmos may be planning the installation of an additional 6–10 miles of 4 NPS SDR 11 PA-11 gas piping. No further details are available on the 6000 ft installation at this time.

Acknowledgments

The following is a partial list of parties acknowledged as participants in the development, testing, installation, documentation, and preparation of this paper on the progress; past, present and today of PA-11 gas piping. Persons: Andre Berry, Dennis Jarnecki, James Mason, Richard Moore, Dr. Eugene Palermo, Richard Sanders, and Pratik Shah. Participating organizations, companies, and utilities: AWG Co., Atmos Energy, Central Plastics, City of Mesa, AZ, Continental Industries, Friatec, George Fischer Sloane, Gas Technology Institute, Kerotest Manufacturing, KWH Pipe (Canada), Ltd., Nicor Gas, Nashville Gas, Nordstrom Valves, Inc., Perfection, Piedmont Gas, Polypipe, PNM, Questar Gas, and R.W. Lyall and Co.⁸

⁸Design pressure = $2 \times \text{HDB}/(\text{DR}-1) \times \text{design factor}$. The MAOP of the system may be limited by code or jurisdiction having authority over the installation.

Richard Wolf¹

Polyamide 12 Natural Gas Distribution Systems Operating at Pressures Greater Than 125 Psig*

ABSTRACT: As a result of the long history of use of polyethylene systems in natural gas distribution systems, the advantages of thermoplastic distribution systems over steel systems are clearly recognized. These advantages include both short-term economic benefits due to advantages in total installed costs and long-term life service benefits due to the excellent corrosion resistance of thermoplastic systems. Historically, a weak point of the thermoplastic systems currently in use commercially are limitations in use due to operating pressure and end use temperature constraints. Initiatives exist within the natural gas distribution sector to identify materials capable of offering the advantages of currently available thermoplastic systems while extending the operating pressure range and end use temperature. Polyamide 12 has been identified as one material capable of meeting these needs. In 2004, a project was initiated by UBE Industries, Ltd. to evaluate the feasibility for use of polyamide 12 in natural gas distribution systems in the United States. The objective of the project was to develop a polyamide 12 natural gas distribution system capable of operating at pressures greater than that imposed by current restrictions and at temperatures above those achieved by commercially available thermoplastic systems. U.S. regulatory restrictions limit the use of thermoplastic systems to a maximum operating pressure of 125 psig. As the project progressed, the scope was expanded to include global activities. The project was broken into two distinct phases. Phase 1, from February 2004 to December 2006, included a comprehensive laboratory evaluation and limited field service use to determine the feasibility for use of

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¹ Polymer Processing Solutions, Inc., Birdsboro, PA 19508.

* Symposium on Plastic Pipe and Fittings: Yesterday, Today, and Tomorrow on 9 November 2009 in Atlanta, GA.

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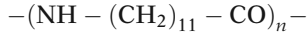
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polyamide 12 natural gas distribution systems operating at pressures greater than 125 psig and elevated temperatures. Long-term strength performance, slow crack growth and rapid crack propagation characteristics, the effect of secondary stresses along with extensive testing to accepted industry test methods were evaluated. Studies were conducted to develop a full set of installation, maintenance, and operating procedures for the use of polyamide 12 systems. Additionally, several small scale private property installations took place as part of the phase 1 project. Based on the positive results from laboratory and field evaluations of polyamide 12 systems, a phase 2 project was developed. The objective of the phase 2 project was to commercialize the use of polyamide 12 systems for use at pressures greater than 125 psig. During this phase of the project, installations are being performed with utilities in North America within their respective operating areas. These installations are being performed under special permits in cooperation with the respective federal and state regulatory bodies. As a separate activity, samples have been removed from the trial installations at specific time intervals and characterized for retention of physical and mechanical properties. The sample removal and evaluation study is an ongoing activity. The results of the project demonstrate that polyamide 12 offers a viable, cost effective alternative to steel pipe for systems operating above the range of currently available thermoplastic systems capability. In terms of both pressure bearing capabilities and high temperature operating performance, polyamide 12 systems extend the range of use for thermoplastic systems in natural gas distribution applications. The paper presents an overview of the laboratory analysis and field installation activities performed in the polyamide 12 gas distribution systems project. The data are presented in the context of compliance to the active ASTM polyamide 12 standards.

Introduction

The natural gas industry and the associated regulatory bodies within the United States clearly recognize the benefits of thermoplastic natural gas transportation systems through their experience with polyethylene. The existing natural gas transportation infrastructure in the United States is incapable of meeting the projected future demand for natural gas consumption. A number of initiatives exist to increase the efficiency and capacity of the existing infrastructure without compromising the public safety. One of these initiatives is to identify materials capable of operating safely and cost effectively at higher pressures without compromising throughput. Polyamide 12 has been identified as one of several materials capable of achieving this objective.

Polyamide 12 belongs to the general class of polymers called polyamides. Polyamides in general and polyamide 12 specifically are characterized by high strength, excellent toughness, excellent abrasion and chemical resistance, and high thermal resistance as compared to conventional polymers such as polyethylene. The chemical formula for polyamide 12 is



The characteristic which gives polyamides their unique properties is the presence of the amide function $-(\text{CONH})-$ in the polymer chain. Differences in properties between polyamides can be attributed to the amide density or the frequency of occurrence of the amide function in the polymer backbone. The presence of the amide function causes intermolecular and intramolecular hydrogen bonding to occur effecting strength, flexibility, impact resistance, moisture absorption characteristics, and thermal properties.

In 2004, a project was initiated to determine the feasibility for use of polyamide 12 in natural gas distribution systems. The main objectives of the project were as follows:

1. Initiate a long-term hydrostatic testing program to generate data to obtain a listing for polyamide 12 in the Plastics Pipe Institute TR-4, HDB/SDB/PDB/MRS Ratings for Thermoplastics Piping Materials or Pipe [1].
2. Develop and initiate a project with the Gas Technology Institute to determine the feasibility for use of polyamide 12 in natural gas distribution systems.
3. Develop and ballot a revision to ASTM D 2513, Standard Specification for Thermoplastic Gas Pressure Pipe, Tubing and Fittings, to include polyamide 12 [2].
4. Participate in various industry forums (American Gas Association, Plastics Pipe Institute, etc.) with the objective of developing an awareness and interest in the use of Polyamide 12 for natural gas distribution systems.

At the present time, the short- and long-term laboratory evaluations have been completed. A series of field evaluations have been completed on both private utility property and in the public right-of-way under special permit. An NPRM has been submitted to the Public Hazardous Materials Safety Administration (PHMSA) for consideration. ASTM D 2513-06a Annex 5 includes polyamide 12 [3]. A set of ISO standards defining requirements for material, pipe, and fittings (ISO 22621 Parts 1–3 [4]) have been approved.

In 2008, in response to an initiative within ASTM Subcommittee F17.60, the standardization development objectives were modified to include the development of a polyamide 12 standard for pipe and fittings for use in natural gas distribution systems independent of ASTM D 2513. ASTM F 2785-09 was approved in September of 2009, and is currently an active standard [5].

The paper presents the results of the polyamide 12 standardization activities and presents the results of testing to the respective standards.

Status

In June 2007, an NPRM to amend Title 49 CFR 192 to allow the use of polyamide 12 systems operating at pressures greater than 125 psig was submitted to PHMSA. Specifically, the NPRM requests a modification to 192.121 allowing the use of a 0.4 design factor with PA12 systems and a modification to 192.123

TABLE 1—PA12 pipe sizes.

Nominal pipe size, in.	Minimum wall thickness, in.	Corresponding SDR values
$\frac{1}{2}$ – $1\frac{1}{2}$	0.090	Variable
2	0.216	11
3	0.259	13.5
4	0.264	13.5
6	0.390	13.5

allowing the use of PA 12 systems to operate above the current 125 psig limitation with the following restrictions:

1. The design pressure does not exceed 250 psig (1728 kPa).
2. The pipe size is a nominal pipe size 6 in. or less; the wall thickness may not be less than that listed in Table 1.

One of the requirements for use of materials in natural gas distribution systems is the establishment of the appropriate material, pipe, and components requirements in an ASTM standard that is incorporated by reference in Title 49 CFR 192. Historically, these requirements were incorporated in annexes of ASTM D 2513. Polyamide 12 requirements are included in Annex 5 of ASTM D 2513-06a. Title 49 CFR 192 recognizes ASTM D 2513-99 [6]. In conversations with PHMSA personnel, it is apparent that PHMSA has no intention of recognizing new editions of ASTM D 2513 past the 1999 edition due to perceived problems with the standard which are unrelated to polyamide 12 systems. Based on input from PHMSA, a project was initiated within the F17.60 Subcommittee to develop a polyamide 12 standard independent of ASTM D2513.

In September 2009, ASTM F 2785-09, Standard Specification for Polyamide 12 Gas Pressure Pipe, Tubing and Fittings [5], was approved and published. The standard defines requirements for materials, pipe, and components for polyamide 12 natural gas distribution systems. Additionally, ASTM D 2785-09 references the following standards for requirements for specific components:

- ASTM F 1733, Standard Specification for Butt Heat Fusion Polyamide (PA) Plastic Fittings for Polyamide (PA) Plastic Pipe and Tubing [7].
- ASTM F 1973, Standard Specification for Factory Assembled Anodeless Risers and Transition Fittings in Polyethylene (PE), Polyamide 11 (PA11), and Polyamide 12 (PA12) Fuel Gas Distribution Systems [8].
- ASTM F 2145, Standard Specification for Polyamide 11 (PA11) and Polyamide 12 (PA12) Mechanical Fittings for Use on Outside Diameter Controlled Polyamide 11 (PA11) and Polyamide 12 (PA12) Pipe and Tubing [9].
- ASTM F 2767, Standard Specification for Electrofusion Type Polyamide 12 Fitting for Outside Diameter Controlled Polyamide 12 (PA12) Pipe and Tubing for Gas Distribution [10].
- ASTM F 2138, Standard Specification for Excess Flow Valves for Natural Gas Service [11].
- ANSI B16.40, Manually Operated Thermoplastic Gas Shutoffs and Valves in Gas Distribution Systems [12].

TABLE 2—*Independent listings for PA12 materials.*

Company Name	Material Designation	Temp, °F	HDB, psi	Grade	Expiration Date
Evonik Degussa	VESTAMID PA12	73	3150	S	Dec. 31, 2013
Evonik Degussa	VESTAMID PA12	140	2000	S	Dec. 31, 2013
UBE America	UBESTA 3035	73	3150	S	Dec. 31, 2011
UBE America	UBESTA 3035	140	2000	S	Dec. 31, 2011
UBE America	UBESTA 3035	180	1600	S	Dec. 31, 2011

Note: Listings for Plastics Pipe Institute TR-4/2009a.

Material Requirements

Polyamide 12 materials suitable for use in natural gas distribution systems as defined by ASTM F 2785 must have a Plastics Pipe Institute (PPI) long-term hydrostatic design stress and hydrostatic design basis rating (HDB) as determined per PPI TR3 [13] and PPI TR4. Materials intended for use at temperatures above 100°F shall have the PPI HDB determined at the specific temperature of use. Table 1.A.18.2 in PPI TR-4 gives independent listings for HDB ratings for polyamide 12 materials (Table 2).

Second, polyamide 12 materials must meet the short-term requirements as defined by ASTM D 4066, Standard Classification System for Nylon Injection and Extrusion Materials (PA) [14] or ASTM D 6779, Standard Classification System for Polyamide Molding and Extrusion Materials (PA) [15], and the long-term requirements as listed in Table 3 of ASTM D 2785-09. The requirements and results of testing performed at the Gas Technology Institute and Jana Laboratories on polyamide 12 are presented in Table 3.

Pipe Requirements

In addition to the standard requirements for dimensional tolerances, ovality, eccentricity, workmanship, etc., the following requirements (see Table 4) specific to polyamide 12 pipe are defined in ASTM F 2785-09.

TABLE 3—*Performance to material requirements of ASTM F 2785-09.*

Property	Test Method	Requirement	Test Result
Relative viscosity	D 789	2.06, dL/g min.	2.70
Melt point	D 3418	170–195°C	177°C
Specific gravity	D 792	1.00–1.06	1.02
Tensile strength	D 638	35 MPa, min	54 MPa
Elongation,(ultimate)	D 638	150%, min	254%
Flexural modulus	D 790	1000 MPa, min	1597 MPa
Charpy or Izod impact	D 256	25 J/M, min	85 J/M
Deflection temperature	D 648	35°C, min at 1.82 MPa	46°C
Moisture “as received”	D 789	0.10%, max	
Long-term hydrostatic strength	D 2837	3150 psi, HDB at 73°F	3150 psi at 73°F

TABLE 4—Performance to requirements of ASTM F 2785-09 for PA12 pipe.

Property	Test Method	Requirement	Test Result
Slow crack growth resistance	F1473 4.8 MPa stress at 80°C	500 h, min	>3000 h
Minimum hydrostatic strength	D1599	3900 psi, min hoop stress	6892 psi
Apparent tensile strength	D2290	3900 psi stress at yield, min	6972 psi
Sustained pressure	D1598	1000 h at 73°F/2800 psi, min	>3000 h

Chemical Resistance—The performance of polyamide 12 to the chemical resistance requirements of ASTM F 2785-09 is as follows (see Table 5).

Rapid Crack Propagation—Requirements for resistance to rapid crack propagation are defined in ASTM F 2785-09 in Subsection 4.4 under material requirements and in Subsection 5.6 under pipe requirements. Testing is to be conducted in accordance with ISO 13478, Thermoplastics pipes for the conveyance of fluids—Determination of resistance to rapid crack propagation (RCP)—Full-scale test (FST) [16], with a modification to the initiation zone temperature as per ISO 22621-Part 1 Annex C. The requirements of F2785-09 state “...The data obtained shall be made available upon request without limitations on disclosure, and shall not subsequently be subject to disclosure limitations when used by others. The values obtained are applicable to all pipes with the wall thickness of the pipe tested and all thinner wall pipes.” Subsection 5.6 defines the requirements further as, “Additional testing for resistance to RCP is required when the wall thickness of the pipe being produced in accordance with this standard exceeds that of the pipe used to establish the resistance to RCP. In these circumstances, additional testing for resistance to failure by RCP in accordance with 6.7 shall be conducted.”

Resistance to rapid crack propagation for polyamide 12 when tested to ISO 1378 with the initiation zone modification as per ISO 22621-Part 1 Annex C is presented in Table 6.

Fitting Requirements

Butt Fusion Joints—Performance requirements for butt fusion joints of polyamide 12 pipe are defined in ASTM 2785-09 Subsection 5.12. At a minimum, the requirements for Minimum Hydrostatic Burst Pressure as per D 1599 [17] and/or Apparent Tensile Strength as per D 2290 [18] apply. Testing on fused polyamide 12 pipe samples indicate that butt fusion joints of polyamide 12 exceed the minimum requirements. Table 7 presents typical properties for polyamide 12 butt fused test specimens.

Butt Heat Fusion Fittings (ASTM F 1733-07)—Butt heat fusion fittings intended for use with polyamide 12 systems must meet the requirements of ASTM F 1733, Specification for Butt Heat Fusion Polyamide(PA) Plastic Fitting

TABLE 5—Performance to the chemical resistance requirements of ASTM F 2785-09.

Chemical	Requirement		Test Results			
	Wt. change, % max	Yield strength, change, % max	Relative viscosity, change, %	Wt. change, %	Yield strength, change, %	Relative viscosity, change, %
Mineral oil	+0.5	±3	0	0	<±3	
Tertiary-butyl mercaptan, 5%	+0.5	±3	0	-1	<±3	
Methanol	+5	±3	+2.3	-31	<±3	
Ethylene glycol	+0.5	±3	0	-1	<±3	
Toulene, 15%	+7	±3	+2.3	-27	<±3	

TABLE 6—Full scale critical pressure for polyamide 12 pipe.

Pipe Size	Full Scale Critical Pressure
110 mm SDR 11 (4 in.)	25–30 bar (363–435 psig)
170 mm SDR 11 (6 in.)	≥25 bar (≥363 psig)

TABLE 7—Typical properties of polyamide 12 butt fusion joints.

Property	Test Method	Test Result
Quick burst	D1599	7324 psi
Tensile stress at yield	D638	5957 psi
Elongation at yield	D638	11%
Elongation at break	D638	121%
LTHS at 80°C/290 psi	D1598	>1000 h

for Polyamide(PA) Plastic Pipe and Tubing. Two performance requirements are defined in ASTM F 1733; short-term rupture strength and performance to sustained pressure testing.

For short-term rupture strength, the minimum short-term rupture strength of the fitting and fused pipe when tested in accordance with ASTM D 1599 must not be less than the minimum short-term rupture strength of the pipe. Testing of polyamide 12 fitting and fused pipe specimens showed the short-term rupture strength to be the same as the unfused pipe. All failures occurred outside of the fusion zone.

The sustained pressure requirement is defined as no failures after 1000 h of testing at a 2800 psi stress at 73°F or 170 h at a 1450 psi stress at 176°F. Samples of polyamide 12 fittings and fused pipe were removed from test at times greater than 1000 h with no evidence of failure.

Transition Fittings and Anodeless Risers (ASTM F 1973-08)—ASTM F 1973-08 defines requirements for polyamide 12 transition fittings and anodeless risers.

TABLE 8—ASTM F 1973-08 requirements for polyamide 12 transition fittings and anodeless risers.

Test	Test Conditions	Requirement
Temperature cycling	(–20 to –140)°F/2.5 h, 10 cycles	Leak free at 1.5 MAOP and 7 psig
Tensile pull	D638 to 25% elongation	No pullout or leakage
Leak test	Pressurize to 1.5 MAOP and 7 psi at 73.4 and –20°F	No leakage
Constant tensile load joint test	F 1588 at 2600 psi stress for 1000 h	No leakage

TABLE 9—ASTM F 2145-09 requirements for polyamide 12 mechanical fittings.

Test	Test Conditions	Requirement
Elevated temperature sustained pressure	D1598 at 176°F/1250 psi hoop stress	No failures after 1000 h
Tensile strength	D638—25% elongation of pipe	No failures
Rotation test	50 ft-lbf torques, min followed by leak testing 2 7 psi and 1.5 MAOP	No failures
Temperature cycling test	(−20 to −140) °F/2.5 h, 10 cycles	Leak free @ 1.5 MAOP and 7 psig
Constant tensile load joint test	F 1588 at 4 psi and MAOP for 1000 hs	No leakage

TABLE 10—ASTM F 2767-09 requirements for polyamide 12 electrofusion fittings.

Test	Test Conditions	Requirements
Minimum hydraulic burst pressure	D1599	>3900 psi fiber stress
Sustained pressure	D1598 at 80°C/1850 psi stress for 170 h	No failures
Tensile strength	D638—25% elongation of pipe	No Failures
Impact resistance	F905— 500 ft lb	No failures
Joint integrity test	D2767 subsection 9.4.1	No failures

Specific performance requirements for temperature cycling, tensile pull, leak testing, and constant tensile load joint testing are outlined. Table 8 presents the performance requirements.

Testing indicates that polyamide 12 transition fittings meet all of the requirements of ASTM F 1973-08.

Mechanical Fittings-(ASTM F 2145-09)—Qualification requirements for polyamide 12 mechanical fittings are defined in ASTM D 2145-09. The requirements are presented in Table 9.

Testing indicates that polyamide 12 mechanical fittings meet all of the requirements of ASTM F 2145-09.

Electrofusion Fittings-(ASTM F 2767-09)—ASTM F2767-09 defines performance requirements for polyamide 12 electrofusion fittings. The requirements are presented in Table 10.

Testing indicates that polyamide 12 electrofusion fittings meet all of the requirements of ASTM F 2767-09. It should be noted that no polyamide 12 saddle type fittings have been evaluated to the requirements of ASTM F 2767-09.

Conclusions

The entire body of information generated during this project’s execution demonstrates the suitability for use of polyamide 12 systems at pressures above the current 125 psig limitations. The ASTM standards that have been developed as a result of the various research activities clearly define the relevant performance characteristics for material, pipe, and components for polyamide 12 systems. Adherence to the requirements of this body of standards will allow the user to safely install, operate, and maintain polyamide 12 natural gas distribution systems.

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Igor Zhadanovsky¹

Plastic Tubing Prospect to Replace Cast Iron Conduit in Steam Heating Systems

ABSTRACT: Steam heating systems become obsolete because of water hammering, noise, high heat loss, installation cost, and poor control. The replacement of metal piping by a coaxial two tube thermoplastic conduit resolves all issues of current steam heating systems along with improvements in heating comfort level, energy efficiency, and maintenance and equipment durability. Polysulfone materials are proposed for the plastic conduit and fluoropolymers for the end connections to the radiators.

KEYWORDS: steam heating system, plastic steam conduit, polysulfone fittings, tubing

“We can’t solve problems by using the same kind of thinking we used when we created them.” –Albert Einstein

Introduction

The first steam heating systems in the United States were put in service in the 1840s, and some of these old timers are still running after more than a hundred years of usage [1]. In 1855, a steam heating system was installed in the White house. The latest news about the White house steam heating system was in 2001 [2], when the system passed a rigorous energy audit by the DOE. Ironically, solar water heating panels installed in 1979 waned out in 19 years and now are in the Smithsonian Institute.

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In Fig. 1, a basic steam heating system set up is shown [3]. No moving parts; simple, low maintenance; and the boiler uses any fuel. Water heated in the boiler becomes gaseous steam, which then rises through pipes and condenses in radiators. Radiators become hot and warm the surrounding air.

The process of heat transfer in a steam system has an important difference from the hot water and forced air heating dominant today. Hot water and forced air employ mostly convection—air heating. Air is a good insulator but a poor heat conductor; besides, warm air tends to rise up and escape from the building. In contrast, infrared radiation energy utilized in steam heating is transferred not through the air but through electromagnetic waves, meaning, the air between the heating unit and the “recipient” does not warm up. A 52 % energy saving was reported for electric radiant panel heating versus electrical baseboard [4]. That is actually an indirect comparison of steam to hot water heating. Besides employing heat with a bigger radiant component, this economy may be explained by the fact that with radiant heat people are typically

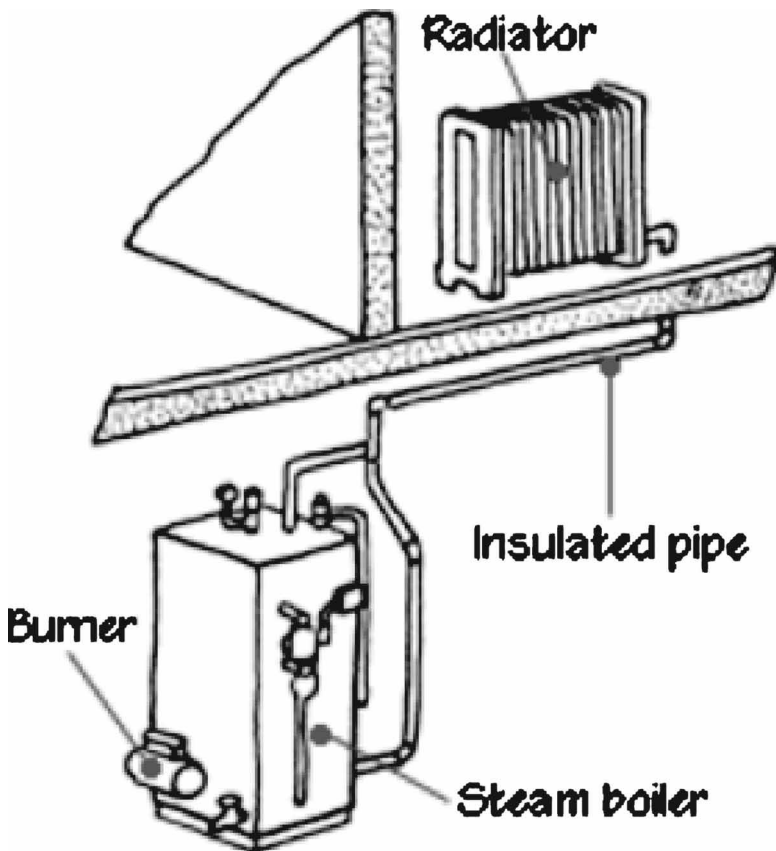


FIG. 1—Basics of steam heating system.

more comfortable at lower air temperatures—as much as 20°F cooler—partially resulting in energy savings.

According to a recent review on steam heating in the United States [5], 1/3 of Americans live in the north eastern corridor, where majority of 50–100 year old houses is heated by boilers, and they number in the millions. Steam is employed heavily in combined heat and power generation for district heating. Typical electricity generation efficiency is 35 %, but it can be increased to 85 % by utilizing waste heat. A recent census found more than 30 000 district heating systems in the United States (some are retrofitted to hot water though), and there are thousands more throughout the world [6]. Additionally, steam based absorption air conditioning helps to defer peak demand from electric system.

Problem Description

Steam heating systems were designed with heavy steel piping and radiators in mind because high pressure was utilized initially (~ 60 psi). Back in the old days, boilers used to explode at a rate of four times per week in America, so in the late 1800s, the “2 psig standard” for maximum residential boiler operating pressure was enforced because of frequent boilers explosions.

Modern boilers and radiators are safer, smaller, and more efficient. It was not possible, though, to replace threaded pipes with copper tubing because soldered joints crack and start leaking when exposed to rapid heating by steam. Expensive in installation, steel pipes are a reason why steam heating systems lost popularity in the last 40–50 years and often are replaced with hot water or forced air when deep retrofitted. Today, thick walls metal threaded piping is still utilized to deliver steam into heavy radiators and reproduces the same hundred year old problems.

- Noise from pipes due to thermal contractions and expansions
- Water hammering—steam up flow meet condensate down flow
- Lower linear velocities help to avoid water hammering but it requires larger pipe diameters—high installation cost
- Bigger pipes require more preheating before steam passage—high heat loss
- Because too much heat is lost in pipes every heating cycle, heat is accumulated in heavy radiators; the typical system pick up factor is 30–50 %
- Slow and inertial heat delivery by heavy radiators causes long temperature hunting cycles and poor temperature control

Solution Proposed

A steam heating system with a tube-in-tube plastic conduit from the boiler to the radiators is proposed *to resolve all listed above steam heating issues* [7]. Steam rises through the inner tube to the radiators and is returned either

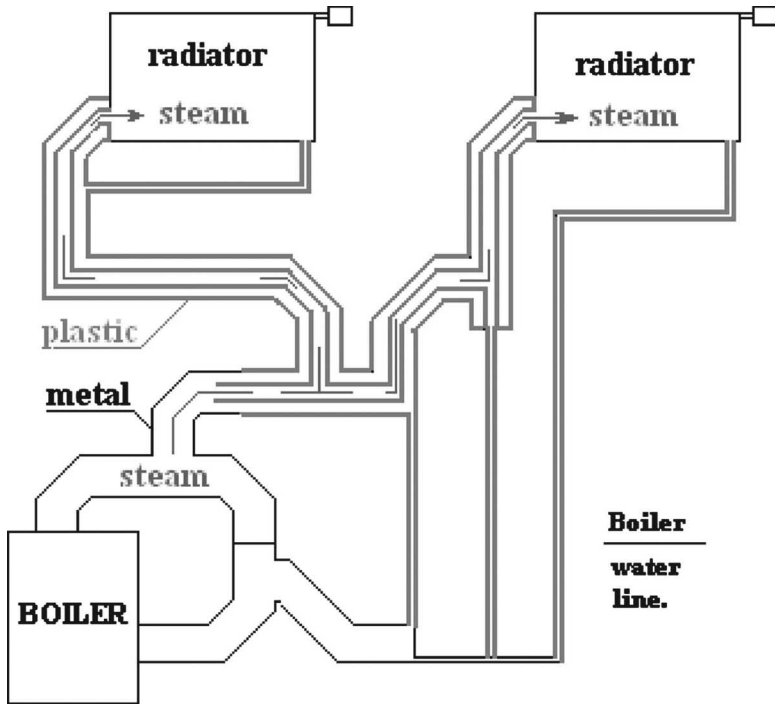


FIG. 2—Steam heating system with tube-in-tube plastic conduit from boiler to radiators.

through the separate line (right radiator) or through the annulus (left one) (Fig. 2) Additional benefits are as follows:

- Lightweight panels are used instead of bulky heavy radiators
- Decreased pressure drop and tube sizes
- Reduced system warm up time—less condensate in the steam supply line is formed to heat and maintain inner tube at steam temperature
- No rust produced (traditionally, steam system requires periodical draining to remove rust particles)
- Reduced materials and installation cost
- New temperature control method per radiator base

Feasibility Study

The feasibility study at the New England Fuel Institute (NEFI), Watertown, MA, proved the tube-in-tube steam conduit concept and confirmed heat loss calculations results.

Tubing and fittings available from local suppliers were utilized. In the prototype, a 23 ft long steam conduit (0.5 in. OD/0.375 in. ID inner Teflon tube, 1.0 in. OD/0.875 in. ID Teflon outer tube) connected a Smith oil boiler to a Runtal

radiator. The outer Teflon tube temperature of 106–115°F was close to preliminary calculations of 92°F. Later, the outer Teflon tube was replaced by a 1 in. cross-linked polyethylene (PEX) tube and tested in four runs, 1–2 h long each.

Teflon tubing was used in the prototype because steam and condensate flows can be seen through the transparent walls. Teflon is an expensive material, but usage might be justified to fish flexible tubing through walls and tight spaces. Available from the hardware store at \$0.50/ft, a 1 in. PEX tube was tested to illustrate an inexpensive alternative to a Teflon outer tube.

Discussion

At the NEFI laboratory, polysulfone fittings were tested for a low pressure steam application. Fittings are rated for hot water applications at a maximum temperature of 180°F at 100 psi. Actually, these fittings' material Acudel® properties exceed the requirements for a steam low pressure system, but the fittings were never tested for an application, which is considered obsolete. With some modifications, existing polysulfone fittings can be adopted for a tube-in-tube steam conduit. In Fig. 3, the conceptual design for double tube fittings is shown. Steam in the inner tube is sealed by an O-ring, while the outer tube is clamped/glued

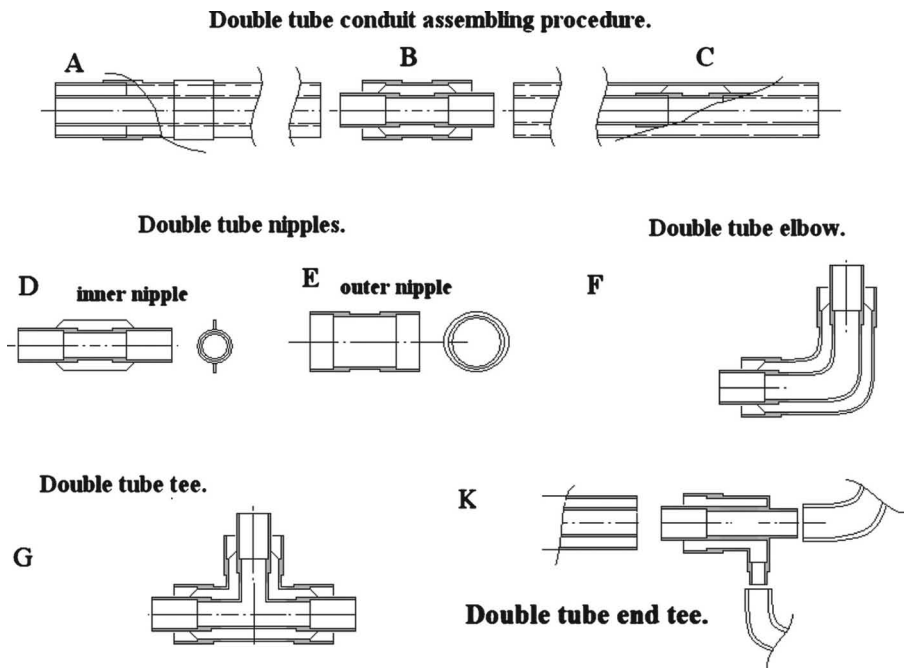


FIG. 3—Tube-in-tube fittings and assembling procedure: [(a)–(c)] Double tube conduit assembling procedure; [(d) and (e)] double tube nipples; (f) double tube elbow; (g) double tube tee; and (k) double tube end tee.

TABLE 1—*Estimation of LTHS for UDEL® P-1700 according to ISO 9080 15.*

Tube Diameter, mm	Hoop Pressure, MPa at 15 psi	LTHS, years	
		15 psi at 120°C	15 psi at 66°C
12.7	0.605	6.1×10^4	6.9×10^7
25.4	0.161	1.2×10^3	7.4×10^5
38.1	1.918	128.5	5.5×10^4
50.8	2.574	26.7	8.9×10^3

to fasten the assembly. To connect the tubes, inner and outer tube nipples (Fig. 3(d) and 3(e)) can be used alone (Fig. 3(a) and 3(c)) or together (Fig. 3(b)). A connection from the polysulfone conduit to the radiators by flexible Teflon or silicon tubing would ease installation and maintenance.

The polysulfone tubing should be custom ordered at a minimum of 1000 ft, so it was not tested yet. However, this solid, relatively inexpensive, and thermally and hydrolytically stable plastic presents a very attractive option. The results of long-term hydrostatic strength (LTHS) calculation for 1 mm thick UDEL® P-1700 tubes (Table 1) indicated that the part's lifetime will probably not be affected by constant hoop stress but design or application related factors.

15 psi at 120°C corresponds to the saturated steam temperature at maximum pressure for a low pressure system. Actually, residential steam heating systems operate at 2 psi at 104°C, so a sufficient safety factor is probably provisioned. 15 psi at 66°C conditions corresponds to the most severe conditions the outer tube will experience if the condensate is returned through the annulus. Again, the outer tube will serve as an additional safety factor should the inner tube sealing fail. Note that estimations are based on results for molded samples and should be verified by tests on extruded tubes.

In Table 2, a metal piping is compared to a polysulfone tube-in-tube conduit. Maximum steam loads for pipes are defined based on a water hammering threshold for horizontal pipes with a counter flow of steam and condensate. For the same load, a polysulfone tubing diameter was determined to keep the pressure drop below 2 psi in a 100 ft long tube. Weight wise, a polysulfone conduit is 15–28 times lighter, the bigger pipe diameter. So the installation will be less labor consuming. What is important is that the metal piping is assembled from nipples of standard length (any non standard length should be threaded on site). Contrary to this, a polysulfone tubing can be easily cut and Lego-like connected by clamps or glue.

Prices are also 1.6–3.4 times less for polysulfone tubes. For the polysulfone double tube, *tees prices* are three to nine times less compared to metal ones, which is an indication of potential savings on fittings.

Calculated *heat loss* for non insulated plastic conduit is, in general, two to three times less than for the same load metal pipe protected by 2 in. thick insulation. So, it is not only heat loss economy but insulating materials and efforts as well. Calculated data were confirmed by experiments at NEFI.

TABLE 2—Comparison of metal pipe and plastic tube-in-tube steam conduit.

Steam Load ^a , lb/h Conduit Material	7		20		65	
	Metal	Polysulfone	Metal	Polysulfone	Metal	Polysulfone
Pipe size, in.	1		1 1/2		2 1/2	
Inner tube OD/ID, in.		0.5/0.4		0.75/0.65		1.0/0.9
Outer tube OD/ID, in.	1.315/1.049	1.0/0.9	1.9/1.61	1.25/1.15	0.87/2.47	1.5/1.4
Weight ^b , lb/126 in. length	17.64	1.24	28.56	1.662	60.826	2.124
Price ^c , \$/126 in. length	42.28	25.36	61.25	38.33	141.44	42.53
Tee price, \$	3.8	1.05	9.04	2.72	29.03	3
Pressure drop ^d , psi/100 ft		1.736		1.04		1.87
Heat loss ^e , BTU/linear ft · h						
No insulation	134.3	10.8 ^f	188	11.8	274.9	12.4
Insulation 1.0 in.	31.6	5.8	41	6.53	55.3	7.14
1.5 in.	26.9		33.2		42.7	
2.0 in.	23.7		28.4		37.9	

^aFor metal pipes, steam load correspond beginning of water hammering in horizontal runs. See Ref. 1, page 89.

^bCast iron pipe weights—<http://www.tubenet.org.uk/specs/asmeshed40.html>, for polysulfone—calculated.

^cCast iron tube/fittings prices are taken from <http://www.mcmaster.com/#>. Polysulfone pricing is based on 2007 quotes for polysulfone injection molding (fittings) and extrusion (tubing).

^dResults are based on <http://www.freecalc.com/gasdiafr.htm>.

^eExperimental data for metal pipe—<http://www.ceere.org/iac/assessment%20tool/ARC2213.html>, for polysulfone—calculated.

^fNEFI experimental data on heat loss from non coaxial tube-in-tube Teflon conduit showing 18–23 BTU/ft h heat loss.

Conclusions

Conducted tests confirmed the possibility of plastic conduit usage for low pressure steam delivery. New technology can improve energy efficiency and comfort level and simplify the schematic for residential and commercial, new and retrofitted buildings, district heating, distributed power applications, and cogeneration heat and power. ASTM D6394-09 [8], Standard Specification for Sulfone Plastics, makes no provisions for polysulfone usage in steam heating applications. New ASTM standards for polysulfone plastic pipe and fitting materials (analogous to ASTM D3350 [9]) and for polysulfone molding and extrusion materials (analogous to ASTM D2116 [10]) are required. Steam heating application specifics for thermoplastic materials should be also included into the following:

- ASTM D 618 [11], Practice for Conditioning Plastics for Testing
- ASTM D883 [12], Terminology Relating to Plastics
- ASTM D1600 [13], Terminology for Abbreviated Terms Relating to Plastics
- ASTM F412-06 [14], Standard Terminology Relating to Plastic Piping Systems

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INSTALLATION

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Specifying Plastic Pipes for Trenchless Applications

ABSTRACT: Plastic pipes are widely used in trenchless applications for both rehabilitation and new construction. What makes a pipe suitable for trenchless installation? What material characteristics and pipe properties should a specifying engineer require from pipe in trenchless applications? Engineers often specify for the same project a number of piping products having significantly different properties. This paper assists the engineer in determining which properties and features are actually important and relevant to a trenchless application and thus which need to be addressed during design. Key properties and features include corrosion resistance, joint reliability, rapid crack propagation resistance, flexibility, static and surge pressure capacity, fatigue resistance, impact strength, pull-in strength, stiffness, safety and ease of tapping and transition to other piping materials, ease of repair, abrasion resistance, cold weather performance, and usage history. The relative importance of these factors to a trenchless application with respect to design, installation, and operation is discussed with an emphasis on the development and preparation of trenchless engineering specifications. In addition, this paper addresses the role that ASTM standards play in defining and quantifying many of these properties. Without standardization it would be hardly possible for a designer to properly select a piping product.

KEYWORDS: plastic pipe, polyethylene pipe, polyvinyl chloride pipe, fused joints, horizontal directional drilling, trenchless technology, rapid crack propagation

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Introduction

In the past 25 years, the introduction and wide acceptance of trenchless technology for new installation and rehabilitation has totally changed the face of the pipeline construction industry: New processes have been introduced. New products have been developed. New businesses have emerged. All these changes are to the advantage of municipal utility companies (water, wastewater, and gas), telecommunication companies, power companies, pipeline companies, and the communities they serve. The driving forces behind trenchless construction include lower construction costs, reduced environmental disturbances, fewer disruptions to business and citizenry, and reduced maintenance costs to roadways.

Underground Construction Magazine's 2008 survey states that the total spending on all types of water and wastewater installations exceeded \$12 billion [1]. It reports that for water installations 24 % of new construction spending was for trenchless construction while 33 % of rehabilitation spending was trenchless. For wastewater installations, as might be expected, only 18 % was spent on new construction using trenchless while 70 % of the rehabilitation dollars were spent on trenchless. Factors causing a higher rate of spending for trenchless rehabilitation of wastewater systems than of water systems include cost considerations as well as the availability and acceptance of more rehabilitation techniques for wastewater systems.

Recognizing the vast investment our country is putting into infrastructure, both new and renewal, it is the obligation of ASTM and other standards writing organizations to develop rigorous standards and requirements for trenchless related products. Such standards protect not only pipeline designers but primarily end-use customers, municipalities, producers, and installers. In addition, ASTM standards give design engineers, particularly those lacking in-depth familiarity with a product, a confident way to specify trenchless products and installation methods. Examples of trenchless standards already developed for use with thermoplastic pipe are ASTM F1962, "Standard Guide for Use of Maxi-Horizontal Directional Drilling for Placement of Polyethylene Pipe or Conduit Under Obstacles, Including River Crossings," ASTM F1804, "Standard Practice for Determining Allowable Tensile load for Polyethylene (PE) Gas Pipe During Pull-Installation," and ASTM F585, "Standard Practice for Insertion of Flexible Polyethylene Pipe Into Existing Sewers" [2–4].

Trenchless Methods

While there are a number of unique and innovative trenchless installation methods, the vast majority of municipal trenchless pipe installations are accomplished using sliplining, pipe bursting, and horizontal directional drilling. This paper will discuss only these techniques and how they relate to the installation and use of polyvinyl chloride (PVC) pipe and high density polyethylene (HDPE) pipe. PVC and HDPE are thermoplastic pipes, meaning that they can be repeatedly softened by heating and hardened by cooling. The author recognizes that other plastic products such as fiberglass reinforced plastic pipe and cured-in-place pipe linings

are used in trenchless applications. Many of the requirements discussed here apply to those products as well, although they are not specifically discussed.

All three of the aforementioned trenchless methods are primarily used as a pull-in installation for thermoplastic pipes. In pull-in, pipe is assembled (joined or fused) into a continuous length on the ground surface. A pulling head is attached to the lead-in end. For bursting or sliplining, a cable is inserted through the host pipe and connected to the pulling head. For directional drilling, steel drill pipe is inserted through the bore hole and connected to the pulling head. The pipe is then pulled into the host pipe or bore. In some cases, a "cartridge" method is used where pipes are inserted into the host pipe or bore as individual pieces, one at a time. The demands of the pull-in installation process are clear: Pipe is pulled along the ground, bent, and deflected downward into an insertion pit, the host pipe, or a bore. The pipe is often pulled over small stones and rocks, curbs, and other obstacles. Existing pipes may have abrasive surfaces such as concrete or be broken into shreds and fragments such as burst cast iron, clay, and concrete pipes. Polyethylene (PE) pipes were one of the first pipes used in trenchless installations and have proven to have the scratch (notch) resistance, strength, and toughness needed for these applications. In fact, PE pipe resin has been engineered and refined for toughness over the years, driven by the critical nature of gas distribution applications, where 90 % to 95 % of pipe currently being installed is PE pipe.

In trenchless applications, the laydown area (where pipe is strung out prior to pull-in) is often not in line with the insertion direction and therefore pipe has to be placed around corners or bends. The more flexibility in the pipe the easier it is to install the pipe. As pipe is placed into an existing pipeline (intact or burst) or a bore hole, the pipe must be lowered from grade. The pipe essentially must deflect through a vertical "S" curve. The more flexible the pipe, the shorter the insertion pit, reducing the cost of trenching. Flexibility increases the pipe's ability to withstand insertion through misalignments or slight route bends in a host pipeline.

During insertion, it is not unusual for the pipe to experience rough handling and impact loads. Heavy construction equipment is on-site and accidental bumps and knocks are to be expected. Pipes are often dropped from forklifts, pushed into trenches, and forced into alignment. These can be rough conditions.

The equipment used to do the pulling is not generally fine-tuned and in the case of directional drilling the rig operator normally does not know the actual pull force applied to the pipe. Drill rigs are powerful and may have thrust (pulling) capacities many times that of the tensile strength of the pipe. The smallest of drilling rigs, mini-horizontal directional drilling rigs, can have pull-in capacity up to 20 000 lbs (89 kN). These are used to pull-in 2 to 12 in. pipes, which have a wide difference in tensile strength. (Pipes are often protected from excessive pulling force by inserting a breakaway link between the pulling head and back reamer. The link contains a pin that will break if the pull force exceeds the pipe's safe pull strength.) Pulling is usually not continuous and occurs with lots of jerks and starts. Dynamic pipe bursting is even more extreme. It applies a dynamic impact load to the existing pipe and to the insertion pipe.

While most utility design engineers and installers understand the rigorous field conditions imposed on pipe by trenchless equipment, how can they measure a pipe's ability to meet these conditions? Does the same pipe that works well for open-cut trench installation work as well for trenchless? Are the risks of using pipe in trenchless the same as in open-cut installations? What physical properties do you look for when selecting pipe to stand up to these rough trenchless installation conditions?

Key Properties and Features for Trenchless Installation

Often a designer will make the assumption that all plastic pipes essentially behave the same, particularly when considering PVC and PE pipes. Therefore a simple comparison of these products based on flow capacity (inside diameter size), pressure rating, and tensile strength is all that is needed. This is a crucial misconception. Other factors often critical to the long term performance of a pipeline must be evaluated and weighed into design decisions. These factors include fatigue strength and toughness properties such as resistance to impact and resistance to rapid crack propagation (RCP). For example, obtaining the highest possible flow capacity for a given size pipe may be desirable; but the small savings in pumping costs gained by using a less tough material may not be worth leaving the utility at risk for a long running split that could be initiated by third-party impact or improper tapping. Offhand, a higher tensile strength seems to offer greater pulling distance, but in practice pulling distance is often limited by project requirements and site considerations, such as rig and exit pit locations and available lay-down space. In many cases, factors such as pipe maneuverability and flexibility have a greater impact on installation costs than pipe tensile strength. Proper design involves considering numerous factors.

At a minimum, the design engineer as well as the owner and the installer will want to evaluate pipe being considered for trenchless installation along the following criteria:

- Corrosion resistance.
- Joint reliability.
- Tensile strength sufficient for pull-in.
- Flexibility.
- Pressure capability and fatigue life.
- Toughness.
 - Impact resistance.
 - Resistance to RCP.
 - Resistance to abrasion and scratch tolerant.
 - Cold weather performance.
- Ease of connection to other pipe materials.
- Convenient repair methods.

Corrosion Resistance

In 2009, the American Society of Civil Engineers gave a grade of D minus to both the U.S. water and wastewater infrastructures [5]. Every day the U.S. has

approximately 700 watermain breaks and loses over 7 billion gallons ($26.6 \times 10^6 \text{ m}^3$) of clean drinking water. Most of the water loss is due to joint leaks and corrosion of traditional metal pipes such as cast and ductile iron. Corrosion in metal pipes takes many forms, oxidation, galvanic corrosion, and chemical attack (acidic soil and sea water) leaving them vulnerable to failure. Plastic pipes have excellent resistance to these forms of corrosion when installed in soil or salt water. For almost every water and wastewater application, corrosion is of negligible concern for PVC and HDPE pipe. (The disappointing part of this is that most cities take water pipe failures in stride. In a recent newspaper report, one city seemed almost proud that they had only 1400 failures a year. The system of sales and purchasing has so much inertia with concrete and metal pipe that it is hard for municipal governments to realize there are better alternatives to deal with corrosion.)

There are limits to plastic pipe's resistance to chemicals. Plastics Pipe Institute's TR-19, "Thermoplastic Piping for the Transport of Chemicals" reports that plastics are susceptible to chemical attack by strong oxidizers. TR-19 is based on immersion and ring tensile testing per ASTM D543, "Standard Practices for Evaluating the Resistance of Plastics to Chemical Reagents" [6,7]. When oxidizers attack plastics, they can break the chemical bonds within the molecule resulting in altered properties of the plastic. Strong oxidizers attack both HDPE and PVC. For all practical purposes, those oxidizers do not exist in storm or sanitary sewers and their existence in drinking water is at a very low concentration making the rate of chemical attack very slow over a broad range of service temperatures. This in part explains the successful use of HDPE and PVC pipes for water distribution. Jana Laboratories issued a report, "Long-Term Performance of Polyethylene Piping Materials in Potable Water Applications," in October 2009 stating that HDPE pipe can last in excess of 100 years under most water quality conditions, service environments, and disinfection techniques. [8]. PE pipe has been successfully used for water pipe in Europe and North America since the late 1950s.

While chemical attack is a process of molecular chain scission, organic liquids may permeate into plastics. Both PVC and HDPE are permeated by organic liquids but have different sensitivity to different organic liquids. While HDPE is more easily permeated than PVC by non-polar hydrocarbons such as gasoline or diesel fuel, PVC is more easily permeated than HDPE by polar solvents such as common paint thinners and dry cleaning fluids.

The Water Research Foundation has documented that the number of reported cases of drinking water contamination by permeation is very low for both HDPE and PVC pipes, in part because of the dilution effect of flow [9]. However, where permeation is a possibility, such as installing pipe in an area of known contaminated soils, it is best to dual contain the pipe or remove the contamination regardless of the pipe type, HDPE or PVC.

Joint Reliability

First and foremost in the selection of pipes for trenchless technology is the prevention of leakage over the service life of the pipe. While gasket-jointed plastic

pipelines have proven to be more reliable than ductile or cast iron water pipelines, on the whole they do not provide the same level of reliability as fusion-jointed pipe. Gasket joints are susceptible to leakage, backflow, shear, over insertion, and pulling apart. A fused pipeline is essentially a continuous pipeline. No thrust blocks are required at the bends. The Water Research Foundation (formerly Awwa-RF) reports that comparative leakage rates are much lower for HDPE pipe than other piping materials [10]. HDPE pipe has been joined by heat fusion for 50 years and recent developments allow some types of PVC pipe to be fused. The fusion procedure for HDPE pipe is standardized, published, and covered in ASTM F2620 [11]. The heat fusion process with HDPE is forgiving and an operator can learn to make reliable joints in a few hours. The gas distribution industry has had an excellent record with fusion with over 500 000 miles (805 000 km) of fused PE pipe installed in North America.

Tensile Strength

Installers normally use pull-in methods for trenchless insertion of thermoplastic pipes. The theoretical maximum insertion distance, whether in a bore or as a lining, is proportional to the pipe's tensile strength. Therefore pipes possessing a higher tensile strength have a longer theoretical maximum insertion distance. Most thermoplastic pipes have the capability in one dimension ratio (DR) or another of being pulled back for at least a mile. For instance, a 4700 ft (1433 m) 28 in. DR13.6 HDPE pipeline was installed in a single pull by directional drilling under the Kinnel Mud Flats, U.K. In the field, insertion distance is rarely controlled by the pipe's tensile strength. Very few applications go a mile; even less go beyond. Other practical factors limiting pull-in distance include laydown area size and location, line curvature, ground conditions, drilling and reaming conditions, bursting limitations, and manhole locations, to name a few.

Safe pull strength, or allowable tensile load as it is also referred to, is specified in at least two ASTM standards for PE pipes. One is ASTM F1804 and the other is ASTM F1962. The fused joint for HDPE pipe has the same tensile strength as the pipe. Therefore, no reduction in the safe pull strength is required when using fused HDPE pipe. Other thermoplastic pipes may have joints that have a lower tensile strength than the pipe.

The safe pull strength formula in ASTM F1962 is based on the safe pull stress (SPS). The Plastics Pipe Institute has published updated values of the SPS in its second edition of the Handbook of Polyethylene Pipe [12]. The SPS value at 73°F for 1 h is generally considered appropriate for directional drilling, as the peak tensile load is only experienced towards the end of the installation and the pulling stress is intermittently applied and reduced during drill rod extraction and removal. The corresponding SPS for PE3608 material is 1350 psi (9.31 MPa) and for PE4710 material is 1400 psi (9.66 MPa).

In theory the achievable pulling distance for a pipe may be estimated by dividing the safe pull strength of the pipe by the frictional force (per foot of pipe length) between the pipe and the surface of sliding. For liner pipe (no bends or curves), the frictional force is the product of the pipe weight per ft and the coefficient of friction (COF) between liner and host pipe.

As directional drilling is one of the more demanding trenchless methods, it merits a longer discussion here. In directional bores, thermoplastic pipe is forced upward to the top of the bore by its buoyancy in the drilling slurry. The surface of sliding is along the top of the borehole. There are primarily three resistance forces during pull-in. One is the frictional resistance between the pipe and the soil/drilling slurry along the top of the bore and between the pipe and ground surface or rollers. The other two are the hydrodynamic drag exerted on the pipe by the drilling slurry and the frictional force intensification at bends due to the Capstan effect (see ASTM F1962.) ASTM F1962 gives formulas for calculating the estimated amount of pullback force required to pull HDPE pipe through a directional bore. Table 1 shows the results of an ASTM F1962 analysis for 28 in. DR11 HDPE pipe in a directional bore in White Bear Lake, Minnesota [13]. The bore length was 3300 ft (1006 m) with a maximum depth of 35 ft (10.7 m). Calculations used a COF of 0.3 inside the bore and 0.5 outside the bore, a conservative drilling slurry weight of 12 lb/gal (14 kN/m³), and assumed ballasting the pipe with water. Figure 1 shows the pipe set to enter the borehole. The lower right corner of the picture shows the barrel reamer used to hold the hole open ahead of the pipe.

When thermoplastic pipe is pulled into a bore, it will stretch. During pulling, creep strain occurs. When the pulling force is no longer applied the pipes will contract. The amount of stretch depends on the pulling force, the pipe's elastic modulus, and the duration of the pull. PVC pipe has a higher elastic modulus than HDPE pipe and therefore will stretch less. For large diameter pipes and long pulls, it is generally a good idea to leave the drill rig connected to the pipe immediately after the pull to allow the pipe to relax and contract. For smaller pipes and short bores, some extra length of the pipe may be pulled to compensate for contraction and to allow disconnection of the pipe from the rig immediately after pull-in.

While tensile strength is important, it should not be the only capability that is considered for determining a pipe's suitability for a trenchless application. While the pipe should have sufficient strength for the pulling distance, equally important is the pipe's ability to withstand the additional stresses that occur when pulled around bends and curves along the alignment and in the laydown (pipe stringing) area.

Flexibility

A primary reason for trenchless installation is to minimize disturbance to the community and to business traffic. Reducing the amount of excavation as well

TABLE 1—28 in. DR11 PE3608 pipe pullback properties.

Pipe	Bore Length	Safe Pull Force	Tensile Strength (Average Wall)	Calculated Pullback Force ASTM F1962
28 in. IPS DR11	3300 ft (1006 m)	275 000 lbs (1223 kN)	690 000 lbs (3071 kN)	140 000 lbs (623 kN)



FIG. 1—HDPE pipe at White Bear Lake, MN.

as the size of the jobsite footprint is desirable. Pipe flexibility plays an important role in achieving this objective. Flexibility is usually measured by the minimum bend radius—the tightest radius of curvature that the pipe can be bent to safely. The factors limiting the minimum bend radius include (1) the outer fiber tensile strain resulting from the combination of curvature and internal pressure and (2) the cross-sectional stability (kinking). For example, a DR17 HDPE pipe, which is commonly used for water mains, has a minimum bend radius equal to 27 times its diameter. The bending limit includes both pipe and fused joints. Thus, a 12 in. DR17 HDPE pipe has a minimum bend radius of 28 ft (8.5 m) which permits it to be installed in a cul-de-sac without the need of any fittings. Fusion-joined PVC pipe has a minimum bend radius equal to 250 times its diameter [14]. A 12 in. DR18 PVC pipe has a minimum bend radius of 275 ft. For trenchless installations pipes with a smaller bend radius allow a smaller jobsite footprint, shorter insertion pits, and more flexibility on uneven ground.

For directional drilling, flexibility provides an important extra safety factor. Drill rod has a radius of curvature between 800 and 1200 times its diameter. The HDPE pipe, for example, is almost always more flexible than the steel drill rod, even though the drill rod is much smaller in diameter thus, eliminating the risk of damaging the pipe by over-bending during pullback. Similarly, the

potential increased drag forces due to pipe stiffness, as the pipe is pulled around bends is minimized. Figure 2 shows typical bend radius values for 24 in. thermoplastic pipe and the 3.6 in. steel drill rod that would typically be used to install it.

Ring Stiffness

PVC pipe has a greater ring stiffness for the same DR than HDPE pipe but, in general, thermoplastic pipes do not possess the same ring stiffness as metal pipes. For trenchless design, ring stiffness often needs to be considered. A number of standards give design information for the installation of thermoplastic pipes in open-cut trenches. These include ASTM D2321 and ASTM D2774 [15,16]. In open-cut applications, the placement and compaction of soil around buried pipes provides support against subsurface external loads. In trenchless applications, particularly directional drilling and pipe bursting, soil compaction is not possible. Even though subsurface loads are considerably smaller than for open-cut applications, trenchless pipes need sufficient stiffness to resist applied external loads. Two standards are available that address short and long-term external load design for PE pipes. They are ASTM F1962 and ASTM F585. These standards help guide the user to selecting the proper DR pipe for the anticipated external loads. External loads include drilling fluid pressure, groundwater pressure, earth loads, and live loads for directional drilling and grout and groundwater pressure for sliplining.

Pressure Capability and Fatigue Life

ASTM standards such as ASTM D2241, "Standard Specification for Poly(Vinyl Chloride) (PVC) Pressure-Rated Pipe (SDR Series)" and ASTM F714, "Standard Specification for Polyethylene (PE) Plastic Pipe (SDR-PR) Based on Outside Diameter" establish pressure rating capabilities [17,18]. For water distribution

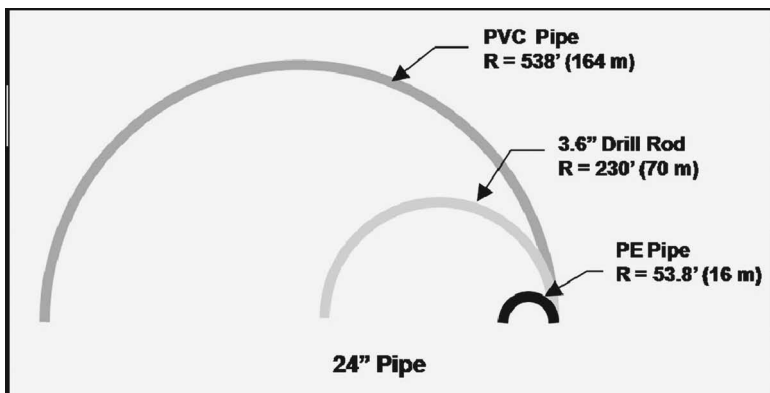


FIG. 2—Bend radius of 24 in. DIPS HDPE and PVC pipe (3.6 in. steel drill rod is typically used for this size pipe).

and transmission applications, AWWA standards for thermoplastic pipes include considerations not only for the steady pumping pressure but also for surge pressures due to changes in flow, such as pump starts and stops and valve openings and closures. Thermoplastic pipes are rated for steady (constant) pressure by applying a design factor and, where applicable, a temperature reduction factor to the material's Hydrostatic Design Basis, which is obtained per ASTM D2837, "Standard Method for Obtaining Hydrostatic Design Basis for Thermoplastic Materials" [19]. As different thermoplastic materials have significantly different capabilities for handling over-pressurizations due to surge, the AWWA method for calculating the allowable surge pressure varies with the piping material. AWWA C900-07, "Polyvinyl Chloride (PVC) Pressure Pipe and Fabricated Fittings, 4 in. Through 12 in. (100 mm Through 300 mm), for Water Transmission and Distribution," requires checking PVC pipes for the surge pressure as well as the frequency of recurring surges [20]. AWWA C900-07 states, "recurring surge pressure may occur millions of times in a piping system's lifetime." Pressure derating from the Pressure Class is required for frequently occurring surges. In some cases, PVC pipes must be derated additionally to increase their fatigue life to an acceptable level. AWWA C901-08, "Polyethylene (PE) Pressure Pipe and Tubing, $\frac{1}{2}$ in. (13 mm) Through 3 in. (76 mm), for Water Service" and AWWA C906-07, "Polyethylene (PE) Pressure Pipe and Fittings, 4 in. (100 mm) Through 63 in. (1,575 mm), For Water Distribution and Transmission," recognize that HDPE pipes have a built in allowance for handling recurring surges up to a combined pumping and surge pressure of 150 % of the pipe's pressure rating [21,22]. HDPE pipe's additional surge capacity above its pressure rating is an important factor for the pipeline designer to recognize. For example, a HDPE DR21 PE4710 pipe or a DR17 PE3608 pipe is safe to use in a system that has a pumping pressure of 100 psi (0.69 MPa) with recurring surges up to a total pressure (pumping plus surge) of 150 psi (1.03 MPa) at or below 80 °F (27 °C).

Recurring surges can have a pernicious affect on some thermoplastic pipes which may lead to premature fracture due to fatigue and therefore for these materials the pipeline designer should determine the fatigue life. HDPE pipe has excellent resistance to fatigue. For example operating at its specified rating with a flow velocity of 4 ft/s (1.2 m/s) or less, a DR17 HDPE pipe can withstand recurring surges at a frequency of one every 15 min for more than 100 years. The following example shows the affect of surge pressure and recurring surges on both HDPE and PVC pipes.

Working Pressure Rating and Fatigue Life Example

The relationship between working pressure rating and fatigue is demonstrated by the following example, which calculates the working pressure rating and the design fatigue life for a PC200 PVC pipe (DR21) and for a PC100 HDPE (DR17) pipe. "On the shelf" the PVC pipe in this example has a stated pressure rating twice that of the HDPE pipe. The example assumes a pipeline operating at 100 psi (0.69 MPa) and 73°F (23°C) with a design velocity of 4 ft/s (1.2 m/s) and with surges occurring at a rate of one every 15 min. This flow rate and surge frequency is not unusual for water distribution lines.

Determine Working Pressure Rating—The working pressure rating of a DR21 PC200 PVC pipe in a system that is flowing at 4 ft/s (1.2 m/s) subject to recurring surge can be calculated using AWWA C900–07 (Section 4.7.1.4.4) as follows:

$$WP = PC \times F_T - P_{RS}$$

$$WP = 200 \text{ psi} - 64 \text{ psi} = 136 \text{ psi} (0.94 \text{ MPa})$$

where:

WP = maximum anticipated working pressure (working pressure rating), psi,

PC = pressure class, psi,

F_T = temperature-compensating multiplier ($F_T = 1$ for 73°F), and

P_{RS} = recurring surge pressure, psi.

A DR17 PC100 HDPE (PE3608) pipe in the same application per AWWA C906–07 (Section 4.6.2) has a working pressure rating of 100 psi (0.69 MPa) and a surge pressure of 45 psi (0.31 MPa) from the 4 ft/s (1.2 m/s) change in flow velocity. No reduction in the pressure rating is required for a surge pressure of 45 psi (0.31 MPa). The combined pumping pressure and surge pressure result in a total pressure during surge equal to 145 psi (1 MPa), which is less than 150 psi (1.03 MPa), the total pressure allowed by AWWA C906 for DR17 HDPE pipe. (Note: Where the velocity is such that the combined pumping and recurring surge pressure exceeds 1.5 times the pipe pressure rating, derating will be required.)

Determine Pipe Design Life for Fatigue—AWWA C900–07 Appendix B gives the method for calculating the number of cycles to failure due to fatigue from recurring surge. The fatigue life of the PC200 PVC pipe in this example can be found by first calculating the mean hoop stress and the stress amplitude for the anticipated surge pressure. For the 100 psi internal pressure, the mean hoop stress is 1000 psi (6.9 MPa) and for the velocity change of 4 ft/s (1.2 m/s), the stress amplitude is 640 psi (4.4 MPa). Using Fig. B.2 in Appendix B, the mean stress and stress amplitude intercept at 1 400 000 cycles to failure. (Different designers may get slightly different values for the number of cycles to failure as this is a graphical solution.) The design fatigue life is found by applying a safety factor to the number of cycles to failure and then calculating a “life” based on the anticipated surge rate. Appendix B of C900–07 uses a safety factor of 2. Applying a safety factor of 2 to 1 400 000 cycles yields a design fatigue life of 700 000 cycles. Based on one surge every 15 min the design life due to fatigue is 20 years for the PC200 PVC pipe operating at 100 psi. A longer design life can be achieved by reducing the operating pressure or using a heavier wall pipe.

Marshall *et al.* reports on cyclic fatigue testing for PVC and HDPE pipes and gives fatigue life graphs (cycles to failure versus stress range) for both materials [23]. The graphs are published in an Information and Guidance Note by Water U.K [24]. Based on the graph for HDPE, DR17 PC100 HDPE pipe operating at 100 psi (0.69 MPa) under the same velocity and frequency of recurring surge in the example has a design life due to fatigue in excess of 100 years.

TABLE 2—Working pressure rating and design fatigue life example summary. Example for operating pressure of 100 psi at 73°F, velocity of 4 ft/s, and 96 surges per day.

Pipe Type	DR	PC	Surge Pressure(psi)	Working Pressure Rating(psi)	Design Fatigue Life(years)
HDPE	17	100	45	100	100
PVC	21	200	64	136	20
PVC	14	305	79	226	56

When fatigue life is considered, the apple-to-apple equivalent to the DR17 HDPE pipe, PC 100, is going to be a heavier wall PVC pipe than a DR21, PC200. In fact, for a velocity of 4 ft/s (1.2 m/s) and 1 surge every 15 min it takes a DR14 PC305 PVC pipe to provide a 50 year design fatigue life based on Appendix B of C900-07. Table 2 summarizes the example.

Toughness

Toughness is an important property for plastic pipes that have to endure the rigorous demands of trenchless installation. In service, toughness is especially important in minimizing damage in case of accidental impact or improper maintenance. With reports that an underground utility is struck once every 60 seconds, the higher the impact resistance the better the chances that damage will not occur. Where impact damage does occur, damage is generally limited to the point of contact or to an area within the vicinity of the point of contact for pipe materials which have a high resistance to RCP.

Impact Resistance

Impact resistance for thermoplastic pipes can be measured by ASTM D256, “Standard Test Methods for Determining the Izod Pendulum Impact Resistance of Plastics” [25]. Table 3 reports the impact strength of PVC and HDPE using ASTM D256 Method A as given by PVC cell classification requirements and laboratory testing of HDPE.

Thermoplastic tensile strength and impact resistance are not necessarily proportional. High tensile strength does not necessarily imply high impact strength. The tensile strength at yield of HDPE ranges from 3200 psi (22.1 MPa)

TABLE 3—Impact strength of PVC and HDPE.

Thermoplastic Material	Izod Impact Strength
PVC (12454)	0.65 ft lbs/in. (34.7 J/m)
HDPE (345464C)	4–5 ft lbs/in. (214–267 J/m)

to 4000 psi (27.6 MPa) depending on the cell classification. Minimum tensile strength of 12454 PVC is 7000 psi (48.3 MPa).

Resistance to Rapid Crack Propagation

Why is impact resistance so important for a thermoplastic pressure pipe? Impact damage can act to initiate RCP. Resistance to RCP is a critical property for fusion-joined thermoplastic pipes. RCP is sometimes referred to as a “fast brittle fracture” and, by the construction industry, as a “linear split.” Once initiated, a crack under sufficient internal pressure can travel the entire length of a pipeline in seconds, splitting or shattering it. Such a crack is driven by the internal water pressure and can travel at very high speeds. For instance, the typical crack speed in PVC is 1550 ft/s (473 m/s) to 1950 ft/s (595 m/s) and in PE is 900 ft/s (275 m/s) [10,26]. Researchers have developed HDPE pipe resins to have a high resistance to RCP because of its use for gas distribution pipe. A Water Research Foundation report on the performance of PE pipes states that “no known incidents of RCP have occurred in PE water pipes in service” [10]. Historically RCP has not been a large concern for the PVC Industry either. RCP events (linear splits) have been reported to occasionally occur during hot tapping of PVC gasket-joined pipes and as a result of damage to PVC gasket-joined pipes. RCP cracks in gasket-joined pipes are limited to a single pipe as they cannot travel through the bell and spigot joint [27]. In fused pipes, RCP cracks are not limited by the joint and can potentially travel large distances. As an example Edwards and Duvall [28] reported on an 1100 ft (336 m) long crack that occurred in a 30 in. DR25 fused PVC pipe that they found to be due to contractor error during the pressure test.

A combination of factors is required for RCP initiation. The pipe must be pressurized or stressed sufficiently to propagate the crack, and a brittle crack must be introduced to the pipe such as through impact, improper tapping, or fatigue. The pressure above which RCP can occur in the presence of a crack is called the “critical pressure.” If the pipe is operating below its critical pressure and a brittle crack occurs it will normally arrest with no sustained RCP. Greenshields and Leever [26] report that the critical pressure for a 100 % water pressurized HDPE line is well above the pressure rating for HDPE pipes. Reporting on PVC pipes manufactured in England, Greenshields and Leever stated that “the 100 percent water pressurized critical pressure falls well below the rated pressure in PVC-U.” This may or may not be representative of current U.S. manufactured PVC pipes resins.

Crack propagation in RCP is arrested if the (pressure) decompression wave in the water travels faster than the crack velocity. In other words, if the water pressure driving the crack drops off faster than the crack can propagate, arrest will occur and thus no sustained RCP can occur. For the PVC pipes manufactured in England that were tested, Greenshields and Leever showed that the wall thickness required to increase the decompression wave speed to a velocity above which RCP cannot occur in water is equal to that of a DR13 pipe or thicker. For the HDPE pipe they tested, the wall thickness was equal to that of a DR29 pipe or thicker [26]. For HDPE pipes the most common DR’s used for

trenchless applications typically have a wall thickness greater than DR29. Therefore, if a crack is formed in HDPE pipe (water filled), RCP cannot occur.

North American water pipe standards (ASTM and AWWA) do not give guidance on a design factor to account for this phenomenon. However, the potential for RCP should be evaluated and appropriate design considerations developed.

Resistance to Abrasion and Scratch Tolerance

Pullback through rocky ground or pull-in during pipe bursting of cast iron or ductile iron (DI) pipes subjects thermoplastic pipes to scratching and gouging. In a double blind study conducted by British Gas on pipe bursting cast iron pipe with PE pipes, it was reported that there was no discernible difference in failure rates for pipes installed with and without a protective sleeve [29]. In metal pipes, abrasive slurries wear off the protective oxide layer and cause erosion-corrosion. Thermoplastic pipes in general have good abrasion resistance. HDPE pipes, for example, are used in conveying slurries such as sand, fly ash, ground limestone, and gypsum and can safely tolerate scratches to a depth of 10 % of their wall thickness [30].

Cold Weather Performance

Special consideration must be given to handling thermoplastic pipe in frigid weather. Thermoplastic pipes tend to become more rigid and their impact strength decreases as the temperature drops. At low temperature the minimum bend radius does not change but it takes more force to bend the pipe, so handling may be more difficult. Dropping pipe such as from the deck of a truck to the ground subzero weather may result in damage. Nonetheless, HDPE pipe, for example, is routinely used in mining applications above the Arctic Circle and can withstand water freezing internally.

Connections to other Pipe Materials

Convenient methods are available for connecting thermoplastic pipe to metal valves, pumps, hydrants, and pipes. For industrial applications such as power or chemical plants, flange connections are the dominant method. A Van Stone style HDPE flange adapter is available for these connections. Historically, cast iron and ductile iron pipes were used in the water market. To more conveniently interconnect with these pipes, PVC water pipe and DIPS sized HDPE pipe are manufactured to the same outside diameter as ductile iron pipe. PVC pipes may be inserted directly into the commonly used mechanical joint (MJ) bell and restrained with an external restraint where required. HDPE pipe connections to Ductile Iron pipe MJ bells are generally accomplished using a HDPE MJ adapter which provides built-in restraint. (See Fig. 3.) These can connect both IPS and DIPS HDPE pipes to Ductile Iron pipe.

DIPS sized HDPE pipe may be connected directly into a DI MJ pipe or a fitting using an insert stiffener in the HDPE pipe and a Megalug or other style restraining device (see Fig. 4). Insert stiffeners for water distribution piping are



FIG. 3—HDPE MJ adapter (on right) and DI elbow.

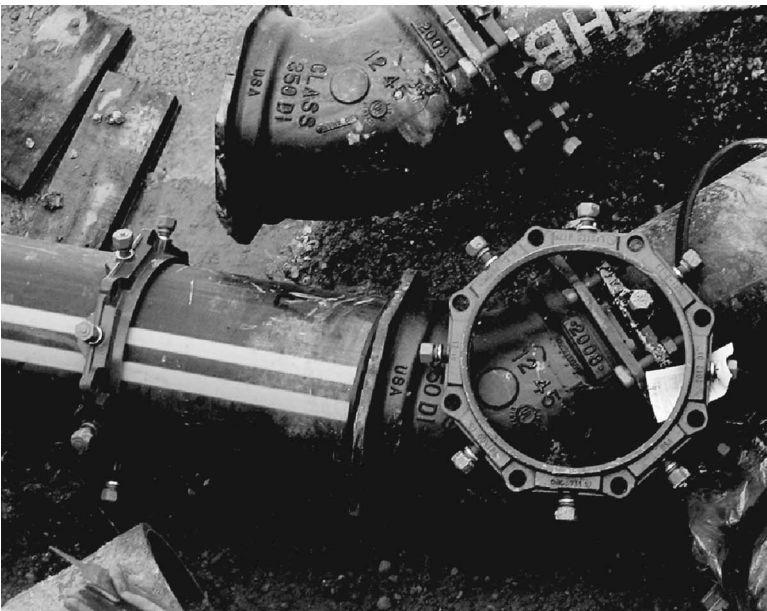


FIG. 4—DIPS HDPE pipe inserted into DI bell with restraints (HDPE pipe has an insert stiffener, not visible in photo).

produced by JCM, Cascade Waterworks, and Romac among others. The best style insert stiffeners are expandable and can compensate for tolerance variations in the inside diameter. Mueller manufactures a restraining and sealing connector known as Aqua-Grip that connects HDPE pipe to DI or PVC pipe for sizes up to 12 in. Similar connectors are produced by other manufacturers such as Smith-Blair and JCM Industries. When using any style mechanical coupling with HDPE or PVC pipe it is important to make sure the coupling manufacturer recommends their fitting for the HDPE or PVC pipe.

HDPE pipe can be cold or hot (under pressure) tapped using piping products presently available. Saddle fusion tapping tees, electrofusion tapping tees, and branch-saddles are readily available to the industry (see Fig. 5.) There are several bolt-on mechanical connections qualified for use with HDPE pipelines as well. With this variety of fittings, tapping is a straightforward procedure.

Full body tapping saddles may be used with HDPE pipe, but saddle manufacturers should be contacted to verify the acceptability of using their tapping saddles and sleeves with HDPE pipe. Generally speaking, most manufacturers

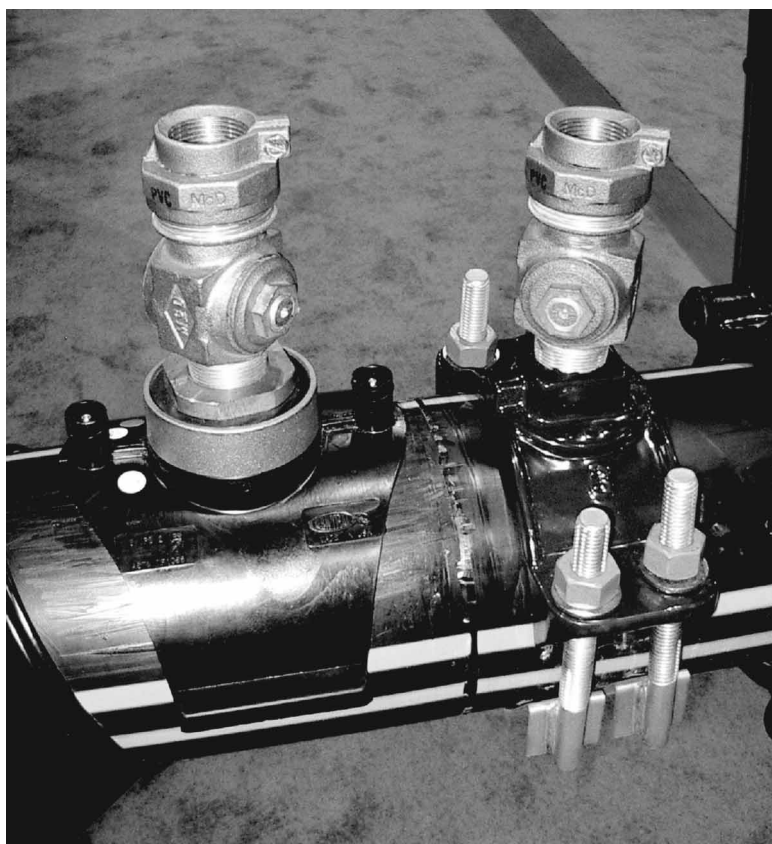


FIG. 5—*Electrofused and mechanical corp stops.*

have saddles and sleeves specifically made for use with HDPE pipe. They are similar to and sometimes the same as those used with PVC pipe. Service saddles often include double straps or extra wide straps and Belleville (spring) washers for use with HDPE pipe so that the tension on the strap remains constant once the nuts are tightened. As for tapping sleeves, some manufacturers such as JCM Industries indicate that as long as it is a full sleeve in accordance with AWWA C110/111, it can be used on HDPE pipe [31,32].

Restraining Transition Connections with PVC and HDPE Pipes

Fundamental mechanics show that when a directionally drilled fusion-joined pipeline is pressurized the pipeline will contract in length, if its ends are inserted into an unrestrained bell or spigot joint. The contraction occurs because no axial thrust force occurs in the fused pipe when it is pressurized. (At least in theory, route curvature and soil friction may provide some restraint from contraction.) As the pipe is pressurized, it expands (ever so slightly) in the hoop direction which causes the contraction in length. The contraction is proportional to the Poisson ratio of the material and the hoop stress. Again, contraction occurs because of the absence of axial thrust force. Depending on the pressure, the pipe modulus, the duration of pressure application, and the length of the bore, the contraction may be sufficient to pull the drilled pipe out of the end connections. The potential for pullout exists for both HDPE and PVC pipes as shown in the example calculation below.

Contraction can be avoided by providing either a concrete anchor at the transition point or restraining the joints of a sufficient number of pipe sections to overcome the contractive forces. The force required to restrain movement of the pipe will be similar to the force required to restrain a 90° elbow. Pipe installed in open-cut trenches are likely less susceptible to this phenomena as the soil friction is sometimes sufficient to prevent significant contraction.

Roark and Young [33] give Eq (2) for the theoretical contraction due to pressurization of an unrestrained straight length of pipe. Actual contraction may be less due to frictional resistance between soil and pipe

$$\Delta y = \frac{qR\mu y}{Et} \quad (2)$$

where:

Δy = change in length (contraction) (in.),

q = internal pressure (psi),

R = radius of inside diameter (in.),

μ = Poisson's ratio,

y = pipeline length (in.),

E = pipe modulus of elasticity (psi), and

t = wall thickness (in.).

For example, Eq (2) gives an initial contraction of 43 in. (1.1 m) for a 2000 ft (610 m) DR21 fusion-joined PVC pipeline pressurized to its rated pressure of 200 psi (1.4 MPa) based on an initial modulus of $E = 400\,000$ psi (2758 MPa)

and Poisson's ratio of 0.38. PVC, such as HDPE, is a viscoelastic material and will creep under constant load. Over time contraction continues. Using a long-term modulus of 200 000 psi (1379 MPa) the calculated long-term contraction equals 86 in. (2.2 m). For the same length of DR17 HDPE pipe at 100 psi (0.69 MPa), the initial contraction is 57 in. (1.4 m) and the calculated long-term contraction is 228 in. (5.8 m). Thus, a restrained connection is needed at the transition of PVC or HDPE pipe into a bell or spigot joint. In addition, a thrust block or additional restraints for bell and spigot joints upstream or down-stream of the fused pipeline may be required.

Repair Methods

Cost effective repair methods are important. Fortunately for thermoplastic pipes, corrosion failures are not likely. Damage is primarily from impact and underground strikes. If the damage is localized to one area of the pipe mechanical tapping saddles or repair clamps may be used. For HDPE pipes electrofusion repair clamps are available in some sizes, thus allowing retention of the integrity of the fused system, see Fig. 6.

In the event that damage to the pipe is too widespread to use a localized repair clamp the damaged section must be removed from the pipeline. The most common method is to cut the section out and replace it with a new short length of pipe using either mechanical or electrofusion couplings. Electrofusion couplings require a dry pipe for assembly. Electrofusion couplings are only made for PE pipes. At the time of this writing, U.S. manufacturers produce them through 28 in. diameter. Larger couplings are available and have been imported for use in the United States. Most of the major manufacturers of mechanical couplings for steel and ductile iron pipes including Smith-Blair, Mueller, JCM Industries, Romac, Dresser, and Ford Meter Box make couplings for HDPE, PVC pipes or both. However, it is important that the particular coupling is designated by the manufacturer to be suited for the thermoplastic pipe with which it will be used. Some mechanical couplings require an insert stiffener in HDPE pipe. Generally a repair spool will be of the same type of pipe as the damaged pipe, however, it is possible use a Ductile Iron pipe repair spool with either HDPE or PVC pipes (see Fig. 4.) See the section "Connections to Other Pipe Materials,"

Summary and Conclusion

Preparing a specification for a trenchless project requires careful consideration of the piping products. Evaluations must be based on more than a simple comparison of flow capacity (inside diameter size) or tensile strength. As shown in this paper, properties not often considered by designers such as fatigue life, bending flexibility, and resistance to RCP are significant in a design assessment of thermoplastic pipes, regardless of the method of installation, whether trenchless or open-cut. Trade-offs of these properties will be required to arrive at a reliable design. For instance, the designer may want to weigh the risk of using a

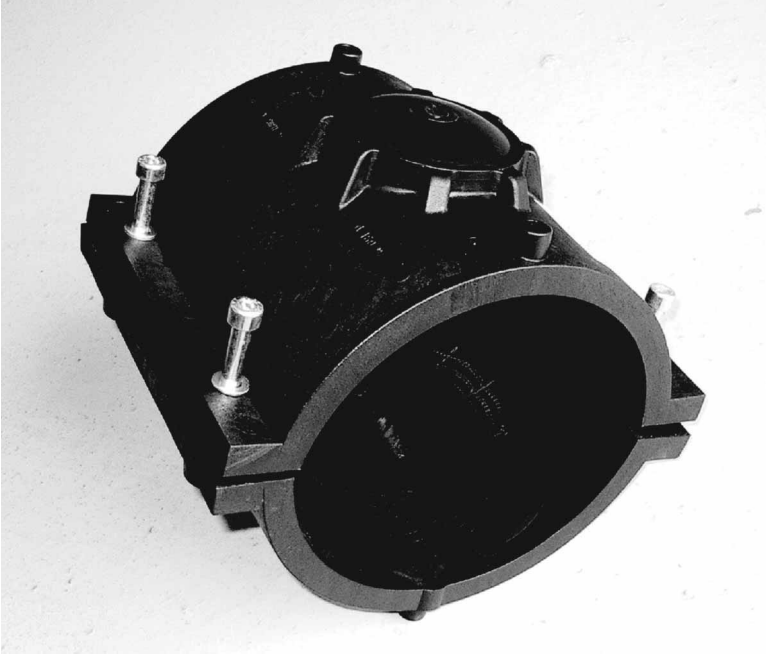


FIG. 6—*Electrofusion repair clamp.*

pipe with low resistance to RCP against the additional pumping cost of using a pipe with a slightly smaller inside diameter pipe but a high resistance to RCP. Higher tensile strength is sometimes helpful during trenchless installation but bending flexibility may be more important.

The designer has to weigh all these factors when considering which products to use. Making sure the owner of a new pipeline is aware of nuances and characteristics of these pipes is also important in assuring that the line is operated and maintained in such a way as to obtain its maximum service life. ASTM and other industry standards provide methods by which the designer can obtain quantitative values for the key engineering parameters as well as measure the differences among varying products. Without standardization it would be difficult for a designer to properly select a piping product.

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Trenchless Case History—University of Denver's Use of High-Performance Restrained-Joint Water Distribution Pipe

ABSTRACT: The benefits of trenchless technology are well understood by all in our industry—more cost-effective and less disruptive to streets, drive-ways, and landscaping. However, what is often less clear to many engineers and city officials faced with a wide selection of materials and processes is just what type of pipe is best suited for a particular trenchless application. This paper will present the engineering considerations involved in selecting thermoplastic water distribution pipe for directional drilling projects, focusing on assembly considerations, structural and flow performance, pressure-handling capacity, joint integrity, and compatibility with existing systems. These concepts are illustrated by the experiences at the University of Denver, where trenchless technology was successfully utilized on an irrigation system in an area where conventional pipe installation techniques were determined to be unsatisfactory.

KEYWORDS: trenchless, horizontal directional drilling, pipebursting, PVC, pipe, restrained-joint, Certa-Lok

University of Denver Waterline Replacement Project

The University of Denver, located south of the city's downtown, decided to construct an irrigation system, which would be connected to two deep wells on campus. They hired general contractor BT Construction of Henderson, CO to

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build a pump station for the wells and install a pipeline to transport water to sprinklers. With school in session year-round and much historic landscaping to protect, it was imperative for BT Construction to use horizontal directional drilling (HDD) to install the pipe.

Horizontal Directional Drilling

HDD is the most commonly used trenchless process for installing water pipes. Traditionally, these applications were reserved for only roadway and river crossings. Today, HDD is employed for a myriad of other applications where the benefits of trenchless installation can be derived. In recent years, developments in the precision of HDD machinery, and specifically, the ability to monitor and steer the pilot bore with high levels of accuracy that maintain line and grade, have also enabled the installation of gravity sewer pipes.

HDD is performed with a drilling rig and involves three essential steps, as illustrated in Fig. 1. First, a pilot bore is created, covering the distance over which the pipe is to be installed. Highly sophisticated electronics enable the drilling rods to be carefully guided and its direction of travel to be monitored. Reamers are then pulled back to obtain a diameter large enough to accommodate the diameter of the pipe; a hole size of $\sim 1.5 \times$ pipe system outside diameter (o.d.) (including couplings, if used) is typically specified. Simultaneously, drilling mud is pumped into the bore to stabilize the boring and to prevent soil collapse. In the final step, the new pipe to be installed is pulled back through the bore.

Construction and Pipe Selection Considerations

BT Construction, Inc. is a general contractor focusing on predominantly municipal water and sanitation projects, including directional boring, auger boring, and major tunneling. Founded in 1980, they now employ 170.

Concerning pipe selection, since HDD was the method of pipe installation, a high-strength restrained-joint pipe that would hold together firmly during the pulling process was required. As is common in many parts of the country, the university's water and wastewater infrastructure is 100 % PVC pipe, so they chose Certa-Lok C900/RJ restrained-joint PVC pipe (Fig. 2) for the project, which the contractor had successfully used in the past.

This pipe was selected because it meets C900 pipe performance standards [1], and matches up to the existing domestic pipe infrastructure, meaning the city would not have to purchase maintenance and repair components for a new pipe material.

Pipe Installation

"Everything had to be directional drilled to maintain pedestrian traffic—we couldn't even close the sidewalks," says Brenden Tippets, Project Manager for BT Construction. "In addition, there are several big, old trees on campus tagged

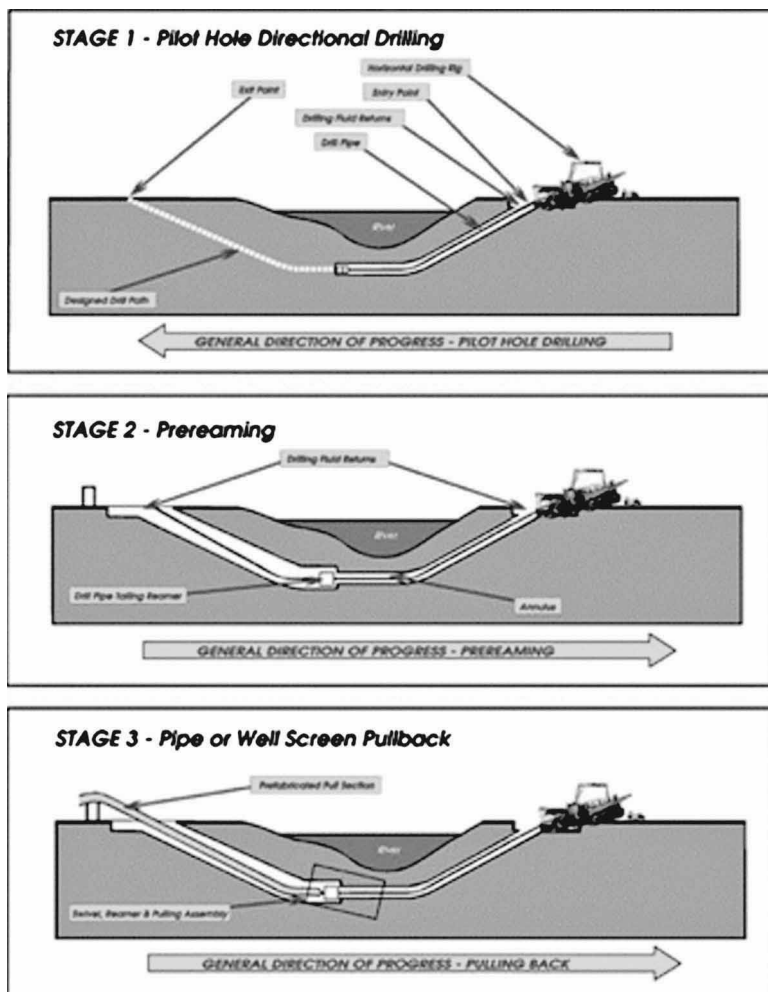


FIG. 1—HDD process.

with the names of their donors from the Eisenhower administration, and we had to be careful not to disturb them.”

For the pipe, BT Construction chose to use cartridge style (segmented) restrained-joint PVC pipe, having had success with it on previous projects. The pipe assembles more quickly than other thermoplastic pipes and would integrate well into the university’s existing water system, also consisting of PVC pipe. The project featured 1000 ft of 8 in. Certa-Lok C900/RJ and 1200 ft of 6 in. C900/RJ—DR18 pipe was used with a pressure rating of 150 psi per C900-97—note that the current C900-07 rating = 235 psi.

“Cartridge style pipe is simple-to-use, since you can put the joints together quickly during the pulls,” Tippetts says. “It’s also the same material as the pipe in



FIG. 2—*Certa-LokTM C900/RJ restrained-joint PVC pipe.*

the existing system, so the university didn't have to adapt and put in new pipe fittings, as they would have with alternate thermoplastic materials."

Certa-Lok pipe is assembled easily by connecting 20 ft lengths of pipe with a PVC coupling as pullback continues. A spline is inserted in the coupling during assembly, which restrains the joints. Elastomeric O-rings provide for reliable pressure integrity. This assembly method is ideal for projects where space is limited, as the pipe does not have to be pre-assembled and strung out in several hundred foot lengths before it can be installed.

Pulling loads and bend radii were well within the allowable limits for the selected pipe, as shown in Table 1.

Project Completion

BT Construction built the pump station and connected it to the two deep wells. Next, the crew of five drilled through silty clay soil, using a Ditch Witch JT-2720 directional drill with 27 000 lb of pullback and 17 000 lb of thrust and a CAT 446 backhoe loader to make excavations and remove dirt. They made eight bores, ranging in length from 300 to 600 ft and averaging in depth between 8 and 12 ft. After each bore was made, the PVC pipe was attached, piece-by-piece, to a

TABLE 1—*Restrained-joint PVC installation limits.*

Size, in.	Pipe o.d.	DR	Tightest Permissible Bend		Maximum Pull-In Force, Tightest Bend Radius, lb	Maximum Pull-In Force, Straight Pull (No Bending), lb
			Radius, ft.	% Change in Pitch per 10 ft		
4	4.800	18	100	10.0	6 700	8 200
6	6.900	18	150	6.7	9 000	12 800
8	9.050	18	200	5.0	18 000	25 200
10	11.100	18	250	4.0	25 600	35 200
12	13.200	18	300	3.3	26 440	41 100
4	4.800	14	100	10.0	8 000	10 300
6	6.900	14	150	6.7	9 300	14 700
8	9.050	14	200	5.0	18 900	28 800
10	11.100	14	250	4.0	24 900	38 300
12	13.200	14	300	3.3	28 300	48 300
16	17.400	25	450	2.2	44 000	68 500
16	17.400	18	450	2.2	44 000	68 500

backreamer and pulled back through the borehole. This assembly method kept the job moving at a steady pace.

One of the biggest challenges of the project came when the pipeline had to go 25 ft below Evans Parkway, a large highway in the middle of the university dividing the north and south campus. The restrained-joint PVC pipe performed well during the difficult pull, which would have been nearly impossible without trenchless construction.

"That was a tough pull, to go that far down and come back up without breaking the pipe or the tracer wire, with a tight deflection in the pipe," Tippetts says. "It took a lot of planning to prepare for it and make sure it was done right. We took about a week to finish that pull."

Once installed, the new irrigation pipeline was connected to the pump station and tied into the existing potable water system, giving the university the option of using either the deep well water or the City of Denver water supply for irrigation, or both. The project was completed successfully in four months and received positive reviews from the university.

"The university was very pleased with the work," Tippetts says. "There would have been no way to open cut this job with the university's requirements for public safety and protection of landscaping. The combination of directional drilling and restrained-joint PVC pipe helped to streamline the process."

Pipe Selection Criteria and Engineering Considerations

The university and the contractor went through a typical process in evaluating the pipe to be used, which includes the following considerations. Generally, the main choices for trenchless construction are thermoplastic (PVC [2] and high-density polyethylene (HDPE)) and metallic pipes.

History and Experience

Metallic pipe has a long history, but research has shown that susceptibility to corrosion is a major liability. Most of the reported leaks in pressurized water systems occur in metal pipes due to the effects of corrosion, wasting billions of gallons of treated water at a significant cost to municipalities and homeowners. Thermoplastic materials, on the other hand, are immune to galvanic corrosion because they are non-conductors, and are immune to electrochemical reactions caused by acids, bases, and salts. PVC is the most widely used thermoplastic material; in sizes 4 in. and larger, about three-fourths of new buried water and sanitary sewer pipe today is PVC.

In the trenchless industry, the first non-metallic, fully corrosion-resistant PVC restrained-joint piping system was introduced in the late 1980s. Restrained-joint PVC pipe is used not only in directional drilling, but also in other demanding applications, including above and below-ground coal and precious-metal mining, agricultural irrigation, and deep water wells.

Compatibility With Existing Systems

For line extensions, including sections to be installed with trenchless techniques, owners and engineers generally prefer to stick with pipe materials they

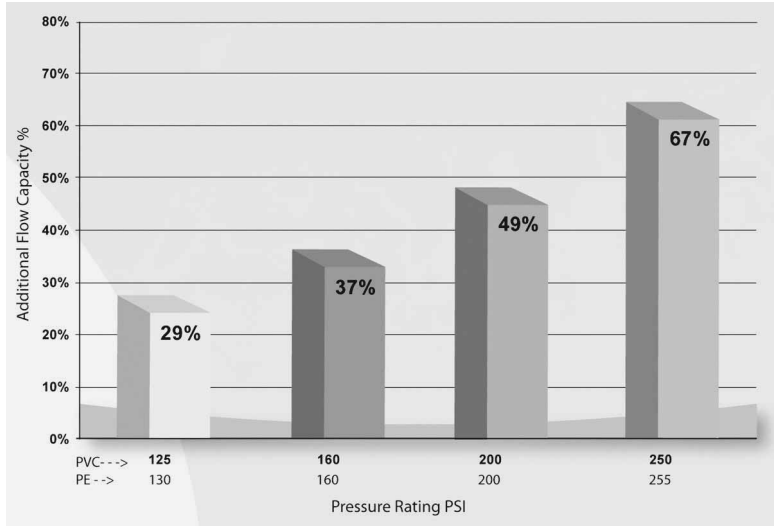


FIG. 3—PVC pipe additional flow capacity compared to HDPE.

have found to offer reliable service. Beyond reliability though, keeping the system exclusively PVC makes line work much easier, as conventional utility components that the city already owns can be used. Water and sewer projects involving other piping materials require that municipalities purchase and stock new equipment and fittings to service the non-standard pipe.

Pressure Handling Capacity

Generally, one of the first considerations in the product evaluation phase of a project is to look at pressure ratings of various plastic pipe products. Using a consistent 2:1 safety factor per American Water Works Association standards, higher-strength PVC requires a much thinner wall than the most common alternative thermoplastic material in order to achieve the same pressure rating [3]. Because of this, a smaller diameter of PVC pipe can often be used, creating real cost savings for the owner. Comparing common sizes and pressure ratings, PVC will have a larger inside diameter, which results in significantly improved flow performance and efficiency, as illustrated in Fig. 3.

Joint Integrity

For most installations, this is rarely a governing factor, as both uncontaminated fused plastic joints and mechanically restrained PVC joints have sufficient strength for typical pulls. Further, when properly assembled, both fused and gasketed restrained-joint systems are leak-free and have the required pressure integrity. The big difference for the contractor was the time and cost involved in fusing joints, which requires special equipment and trained operators. Furthermore, the entire string of fused pipe needs to be joined before pullback starts. In

many urban and crowded areas, there simply is no room to do this. When there is room, several hundred feet of streets and driveways may wind up getting blocked. With today's mechanical spline-lock joints, assembly of 20 ft sections takes place as the pullback continues, which causes considerably less traffic disruption, and requires a much smaller staging area. This feature is particularly important where the installation method involves lowering a pipe into a deep, narrow pit. In these cases, a gasketed restrained-joint segmented pipe will almost always be the more practical option.

Agency Approvals

While not a consideration for the university on an irrigation system, many other water systems have hydrants, so approval of the pipe for use in fire-protection systems is critical. Engineers should make sure that the pipe being specified has been tested and is approved by a recognized certifying agency (typically Underwriters Laboratories). With a proven track record in HDD, restrained-joint PVC pipe is now being successfully used in another important trenchless construction technique, static pipebursting. Pipebursting is a rehabilitation method for replacing both pressure and gravity lines with a new pipe, and involves the breaking of an existing pipeline by brittle fracture, using mechanically applied force from within. While the deteriorated pipe fragments are forced into the surrounding ground, a new pipe of the same or larger diameter is pulled in to replace the original pipe. Static pipe bursting is performed through the insertion of a conically shaped bursting head into a deteriorated pipe, causing it to shatter by hydraulic action. The diameter of the pipe being burst typically ranges from 2 to 30 in., although pipes of larger diameters can be burst. Pipe bursting is commonly performed size-for-size or one size above the diameter of the existing pipe. Larger upsize (up to three pipe sizes) has been successful, but the larger the pipe upsizing, the greater the force required to burst the existing pipe and to pull the new pipe, and the greater the potential for ground movement (upheave).

Conclusion

Cartridge style PVC pipe continues to increase in popularity due to its many features and benefits, with new versions now reaching the market. Owners should carefully evaluate all options, with a special eye on keeping systems and components completely non-metallic to minimize corrosion concerns.

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Richard (Bo) Botteicher¹ and Samuel T. Ariaratnam²

Unique Calibration Case Study for Predictive Model of Installation Loads for Directional Drilled Fusible PVC Pipe

ABSTRACT: The nature of the horizontal directional drilling (HDD) process necessitates the use of pipe products that are capable of being pulled into final alignment via a tensile force, as opposed to being pushed or jacked by segmental means. While steel and high density polyethylene pipes have been historically used with HDD methodology, a fusible polyvinylchloride pipe (FPVCP) product has been one relatively recent addition to the materials available for the installation of water, wastewater, pressure, and non-pressure infrastructure. Maximum safe pull load criteria have been developed for this material, and recent research was focused on aligning current predictive pull force modeling with this material for use in HDD application. Recent work has presented the predictive model and general validation of its use with FPVCP. One particular set of monitored HDD installations that was gathered during the field validation stage of the research provided unique insight into the use of the predictive model to calculate the required pulling force for use in HDD installation methodology. An overview of the model background, field validation study, and specific case study will be reviewed. Specific conclusions regarding this case study relate to the importance of drilling fluid accuracy in the model.

KEYWORDS: horizontal directional drilling, PVC pipe, pull force, fusion, modeling

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Introduction

The water and wastewater infrastructure community, including engineers, contractors, utility owners, and operators, must maintain a constant level of knowledge of emerging trends and methods for infrastructure installation and rehabilitation. One such trend has been the movement to adopt alternate “trenchless” or “minimal excavation” methods of pipeline construction, which includes horizontal directional drilling (HDD). HDD provides benefits that traditional direct bury construction methods cannot. These include less impact to surface activities, reduced socio-political costs (i.e., traffic and pedestrian concerns), reduced direct construction costs (i.e., rehabilitation of streetscapes and surface features), and making challenging projects possible (i.e., under major highways, river crossings, or railroad tracks). The scope of municipal projects being constructed using HDD include potable water, force mains, reclaimed water, and on-grade gravity sewer installations.

The HDD process requires that a pipe product be capable of installation via a tensile force (i.e., pulled in place). This requires a joining methodology for the pipe that allows for high tensile loading against such restraint. Steel pipe with welded joints and high density polyethylene (HDPE) pipe joined by thermal butt fusion are two pipe materials that have traditionally been favored by the HDD industry due to their fully restrained, low-profile, and non-mechanical joining methods. A relatively new product, Fusible polyvinylchloride pipe (FPVCP), which is a PVC plastic piping product, utilizes a butt fusion joining procedure, thereby creating another HDD friendly pipe material for use in the water and wastewater industries.

A research project was undertaken to create and validate a model for use in predicting pulling loads for FPVCP installed using the HDD methodology [1]. That initiative not only included the creation of a predictive model or correlation of existing predictive models for FPVCP pullback loading during HDD installations, but it also involved the field validation of such a model by monitoring a diverse set of commercial installations using an inline pull load measuring device (Fig. 1). The determination and field validation of this predictive model using data captured during commercial installations has been summarized and is discussed in this paper.

This paper focuses on a set of HDD installations that were part of the same project located in the City and County of Broomfield, CO, in which an identical FPVCP product was used for three installations as part of a larger project. Predictive calculations from these models are compared and discussed with conclusions offered regarding the use of predictive pull force models for HDD pulling loads.

Horizontal Directional Drilling Installation Methodology

HDD technology enables the installation of conduits and pipelines ranging from 2 in. (50 mm) to 60 in. (1500 mm) in diameter over extensive distances, up to 10 000 ft (3 km), with minimum need for open-cut surface excavation. The advantages presented by directional drilling, such as reducing disturbance to traffic and businesses and elimination of restoration costs, make this

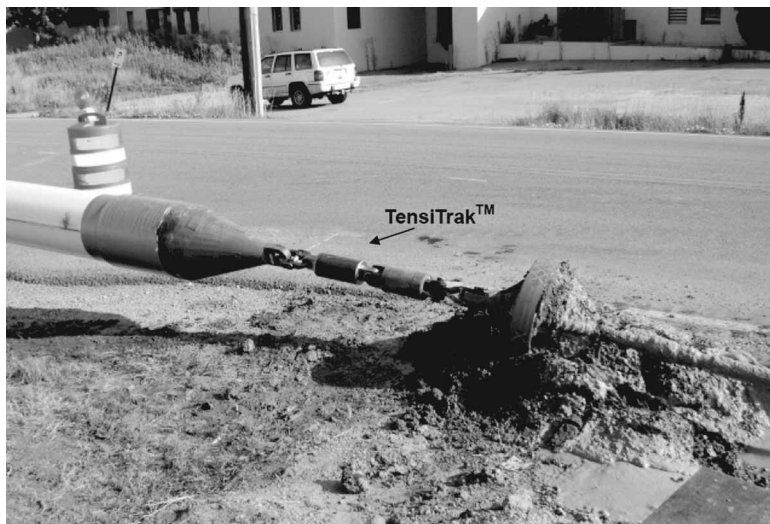


FIG. 1—Example and position of the tensile load monitoring equipment in the drill string, between the swivel and pipe pulling head, used to monitor and collect installation data.

technology an attractive alternative to open-cut excavation for the installation of pipelines and conduits in congested urban areas. As previously mentioned, the technology is particularly suitable for projects that involve crossings of rivers, traffic corridors, or the installation of pipeline products beneath existing structures. In most cases, HDD is selected because it provides a comparable or lower total cost alternative to open-cut construction [2].

The horizontal directional drill rig provides the torque, thrust, and pullback force required to drive the drill string. The drill drive assembly resides on a carriage that travels under hydraulic power along the frame of the drill rig. The bore is launched from the surface and the pilot bore proceeds downward at an angle until the required depth is reached (Fig. 2). Then, the path of the bore is gradually brought to the horizontal and the bore head is steered to the designated exit point where it is brought to the surface along a curved bore path. A

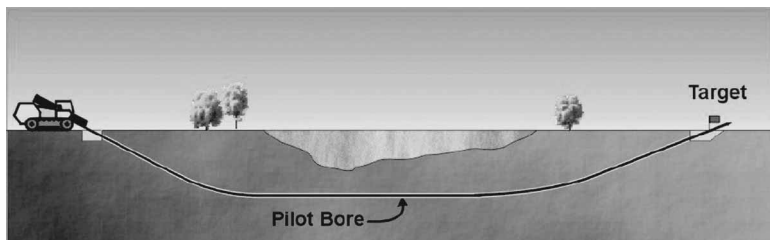


FIG. 2—Diagram of pilot bore creation, from drill rig to target, passing under typical obstacle.

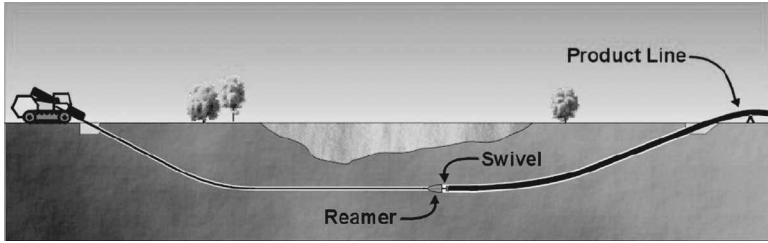


FIG. 3—Diagram of pull back procedure and pipe installation after pilot bore and reaming operations have been completed, showing pipe passing back under obstacle from target to drill rig.

directional monitoring device located near the head of the drill string is used to track the position of the drill head. After the pilot string breaks the surface at the exit location, the bit is removed from the drill string and replaced with a back-reamer. The pilot hole is then back-reamed, enlarging the hole to the desired diameter while simultaneously pulling back the product pipe behind the reamer, as illustrated in Fig. 3. During the boring process, drilling fluid is injected under pressure ahead of the advancing bit. Drilling fluid is composed of a carrier fluid (typically water) and solids (clay or polymer). The carrier fluid carries the solids down the borehole, creating a “mud cake” along the perimeter of the borehole, thereby stabilizing the borehole and reducing friction during the pullback operation. Drilling fluids also function as coolant for the electronics at the drill head, suspension and transport of drill cuttings to the surface, and to reduce the shear strength of the soil to enable easier displacement during the pullback operation. During the drilling process, the bore path is traced by interpreting signals sent by electronic sensors located near the drill head. At any stage along the drilling path, the operator may obtain information regarding the position, depth, and orientation of the drilling tool, therefore allowing the navigation of the drill head to its target [3,4].

Prior Research Involving Horizontal Directional Drilling Predictive Models

Installations of municipal underground infrastructure systems using HDD have seen a dramatic increase in recent years. Although there are currently no national standards regarding HDD installations for any pipe material, such pipeline installations are becoming more and more common and arguably is the fastest growing trenchless construction method today [5]. Industry groups have been promoting good practice guidelines as this installation methodology gains acceptance. A scholarly approach to evaluation of design methodology is important to ensure that basic engineering principles are met during installation and to satisfy the requirements of designers and municipal agencies.

Predictive equations for required stress loading during proposed HDD installations have been developed and published specific to the pipe product materials being installed. For example, a design criterion has been established for HDD

installation of steel pipe, which has been primarily used for oil and gas pipeline application 6. Similarly, HDPE pipe is also a common material for use with HDD installations. The ASTM has a published Design Standard F1962-05 for pull force loading specific to HDPE pipe [4]. Additional publications regarding material specifications for HDD applications can be found in literature [2,3,7].

Duyvestyn [8], Slavin [9], Cheng and Polak [10], Petroff [11], Baumert *et al.* [12], Puckett [13], Francis *et al.* [14], Baumert and Allouche [15], Gelinas *et al.* [16], and Knight *et al.* [17] have all conducted various research efforts related to predicting pull loads on HDD installations. All of this prior research focused on HDPE pipe. It should be noted that the pullback force for any product pipe to be installed depends on a number of factors, including the size of the borehole, type of slurry, density of slurry, soil conditions, type of pipe material, and whether the pipe is filled with any ballast material. A review of existing literature found no prior research related to predictive modeling of HDD installations employing FPVCP.

For the research presented in this paper, the ASTM F1962-05 equations for prediction of pull loads were used as reference. A majority of contractors depend on their experience in selecting the equipment and drilling fluid composition. The prediction of pull loads (with a factor of safety) can aid contractors in determining the most economically feasible equipment for a given project. The development of analytical procedures to determine the appropriate HDD design characteristics of the FPVCP is imperative to project success. An analytical procedure for determining safe pull loads would aid designers and contractors involved in HDD installations.

Technical Background of Fusible Polyvinylchloride Pipe for Horizontal Directional Drilling Installations

FPVCP relies on the tensile strength of the PVC pipe material since thermal butt fusion is the joining methodology employed. FPVCP contains no mechanical coupler or separate gasket as part of the joining process. Much like the predominant method for joining polyethylene pipe, thermal butt fusion for PVC involves aligning two plain-end pieces of pipe, heating the ends until a fusible bead of material is created, then pressing the two ends together and holding them under pressure until the newly created joint cools back down to ambient conditions. In regards to HDD installations, this joint is advantageous due to the fact that the full tensile capacities of the PVC plastic are realized, and the low profile of the joint does not complicate the insertion process or create a need for a larger borehole than that of the pipe itself. Post-extrusion fusion of the PVC material that allows for a joint that carries the same tensile capacity as the plastic itself is created by a specific formulation for the PVC and a specific fusion process used to join separate lengths together.

Fusible Polyvinylchloride Pipe for Horizontal Directional Drilling installations

The FPVCP used carries a pipe supplier recommended safe pulling force based on the tensile capacity of the PVC and an appropriate factor of safety. Testing

TABLE 1—*Material properties of PVC for use with model.*

Property	PVC
Density	1.4
Tensile strength	Short duration loading = 7000 psi
Safe pull stress	Short duration = 2800 psi 24 h duration = 2800 psi
Young's modulus	400 000 psi
Specific gravity	1.39–1.42
Poisson's ratio	0.38

has been used to verify the tensile capacity of the fusion joint and the pipe. A safe allowable tensile stress for the PVC of 2800 psi (19.3 MPa) is set based on a 7000 psi (48.26 MPa) material tensile strength with a safety factor of 2.5. A calculated maximum recommended safe pull force can then be created for each pipe section by multiplying this safe pull stress by the cross-sectional area of the pipe being installed.

A pipe supplier based minimum recommended bend radius is also advocated and is ~ 250 times the outside diameter of the pipe, which is consistent with other PVC bend radius guidance [18]. This also contains a 2.5 safety factor and contemplates the axial stress encountered in the bending of the pipe cross-section along with other axial stresses included in the pipe insertion.

A positive, through-bolt, smooth shank pull head was also developed specifically for FPVCP installation, including HDD insertion. This pull head design allows for the full recommended safe allowable pull force for each size of pipe to be realized. Also containing an appropriate safety factor, this assures that the recommended tensile capability of the pipe section may be utilized in a HDD insertion.

Determination of Fusible Polyvinylchloride Pipe Model from Existing Predictive Models

The design approach in ASTM F1962-05, which deals specifically with HDPE product pipe, was used as a foundation for the prediction of pulling loads for the projects monitored [4]. As this standard deals with another, albeit different, thermoplastic pipe for use in HDD and the joining methodology between HDPE and FPVCP are very similar, ASTM F1962-05 was thought to provide the best starting point in terms of theory; however, material properties specific to PVC pipe were used in the model. Table 1 presents the specific material property values for PVC that are critical to the methodology presented by ASTM F1962-05 and that were used in the PVC model.

The equations for predicting the tensile loading on the pipe during pullback are performed for four critical installation points A, B, C, and D along a typical bore path, as illustrated in Fig. 4. Suggested design values for the frictional coefficients ν_a and ν_b are 0.5 and 0.3 according to the ASTM F1962-05 standards. A value of 0.1 is suggested for the coefficient of friction if the pipe is placed on a

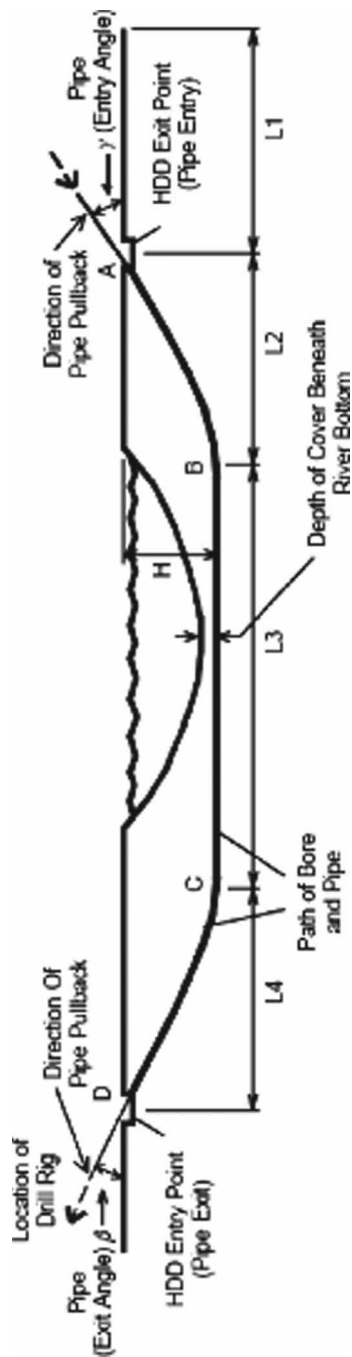


FIG. 4—HDD bore layout in terms of pull back force estimation, the four critical installation points, and the variables that affect the predicted loading [4].

sufficient number, size, and functioning set of pipe rollers. These values were kept constant for FPVC modeling based on the similar surface frictional properties between FPVCP and HDPE. ASTM F1962-05 equations are used to determine the pulling loads at discrete points A, B, C, and D along the borepath, as shown in Eqs (1)–(4) and illustrated in Fig. 4

$$T_A = \exp(v_a \alpha) [v_a w_a (L_1 + L_2 + L_3 + L_4)] \quad (1)$$

$$T_B = \exp(v_b \alpha) [T_A + n_b |w_b| L_2 + w_b H - v_a w_a L_2 \exp(v_a \alpha)] \quad (2)$$

$$T_C = T_B + v_b |w_b| L_3 - \exp(v_b \alpha) [v_a w_a L_3 \exp(v_a \alpha)] \quad (3)$$

$$T_D = \exp(v_b \beta) \{T_C + v_b |w_b| L_4 - w_b H - \exp(v_a \alpha) [v_a w_a L_4 \exp(v_a \alpha)]\} \quad (4)$$

where:

T_A = pull force on pipe at point A, lb (N),

T_B = pull force on pipe at point B, lb (N),

T_C = pull force on pipe at point C, lb (N),

T_D = pull force on pipe at point D, lb (N),

L_1 = additional length of pipe required for handling and thermal contraction, ft (m),

L_2 = horizontal distance to achieve desired depth, ft (m),

L_3 = additional distance traversed at desired depth, ft (m),

L_4 = horizontal distance to rise to surface, ft (m),

H = depth of bore from ground surface, ft (m),

$\exp(x) = e^x$, where e = natural logarithm base ($e = 2.71828$),

v_a = coefficient of friction applicable at the surface before the pipe entering bore hole,

v_b = coefficient of friction applicable within the lubricated borehole or after the wet pipe exits,

w_a = weight of empty pipe, lb/ft (N/m),

w_b = net upward buoyant force on pipe in borehole, lb/ft (N/m),

α = borehole angle at pipe entry (or HDD exit, at side opposite drill rig), radians, and

β = borehole angle at pipe exit (or HDD entry, at same side as drill rig), radians

During pullback, the movement of the pipe is resisted by the drag force of the drilling fluid. Again, taken from the ASTM F1962-05 guidance, the hydrokinetic force (Eq (5)) depends primarily on the drilling slurry, slurry flow rate, the borehole diameter, and the product pipe outer diameter. This must be added to the maximum calculated pull load to obtain the “true” maximum prediction at any point in the pullback operation

$$T_{HK} = q^* (\pi/8)^* (D_H^2 - OD^2) \quad (5)$$

where:

q = hydrokinetic pressure (= 10 psi or 0.069 MPa),

D_H = borehole diameter, and

OD = pipe outside diameter.

Case Study Application: Broomfield, CO, Sheriden Reuse Extension Project (Phase II)

A unique opportunity to study the proposed theoretical model for FPVCP was presented during a particular project for the City and County of Broomfield, CO (Broomfield). As part of the installation of a longer pipeline, several HDD insertions were to be installed end to end. With the monitoring of these insertions, many of the variables normally encountered from project to project were eliminated. This enabled the model to be scrutinized for accuracy, consistency, and precision; and otherwise provided a unique validation of the results across three very similar installations.

Background of the Reuse System in Broomfield, CO

The current City and County of Broomfield, CO sits today on what used to be an expanse of rolling farms between the City of Boulder and the ever increasing Denver-Metropolitan area in the Front Range of the Colorado Rocky Mountains. Source water and water conservation is of paramount importance in the semi-arid high-desert of Colorado. In the late 1980s, Broomfield embarked on a community wide plan to implement reuse irrigation water for public parks and green space. This began as a main east-west artery of reuse water distribution and storage, which the community has been expanding ever since. Broomfield is now directing their attention to providing reuse water distribution to newer areas of development. Massive expansion and growth in the Northeastern quadrant of the community is where they are focusing most of their reuse expansion going forward.

Illustrated along the bottom of Fig. 5 is the current system that is in operation today [19]. Future expansion includes piping that will deliver water to the Northeast Quadrant for the ultimate system expansion (shown with gray hash marks), estimated to be completed in 2032. The extension being installed currently will bring online about another 1 000 000 ft³ (~28 000 m³) of demand per year. The original system will also add another 1 350 000 ft³ (~38 000 m³) of demand, which brings the total usage for reuse water in the system in the next several years to 5 250 000 ft³ (~148 500 m³). In the long term, it is estimated that the Northeast Quadrant expansion will take the total system demand up to 7 670 000 ft³ (~217 000 m³) annually. This represents about half of the current potable water demand that Broomfield uses per year, and highlights the benefit of such a system in an area where source water for potable consumption is a limited resource [20].

In the summer of 2007, Broomfield embarked on its Sheridan Reuse Extension project to extend reuse distribution along Sheridan Boulevard between 136th and 144th avenues. They investigated using new technology and techniques to install this large diameter pipeline in an already developed area. Having some experience with HDD installation methodologies, utility engineers evaluated the feasibility of avoiding the construction impact and social cost of direct bury construction on such a major thoroughfare by drilling under everything. As part of this phase, the contractor crossed one major intersection and a stretch of high traffic roadway with two HDD installations.

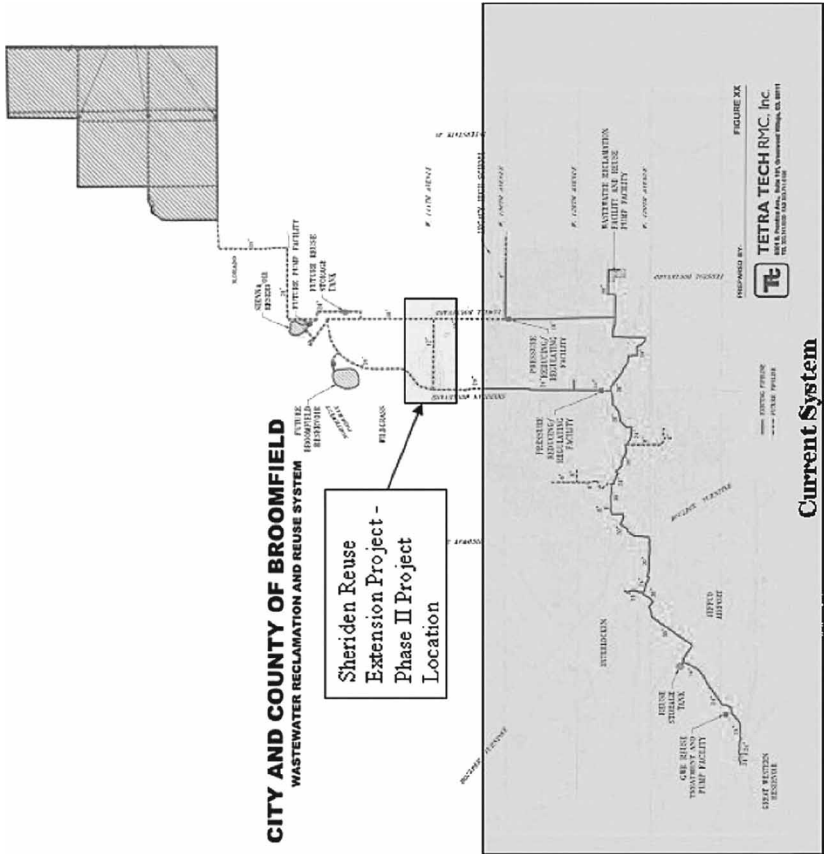


FIG. 5—City and County of Broomfield, CO, overview of reuse irrigation distribution system 19 with current system and project location, including the location of the three subject installations where pull force information was gathered under similar conditions, highlighted.

TABLE 2—Similar HDD parameters for the Sheridan Reuse Extension Project (Phase II).

Parameter	Description
Drilling contractor	BTrenchless, a division of BT Construction, Henderson, CO
Drill rig	DitchWitch JT4020 (i.e., 40 000 lb or 178 kN thrust/pullback)
Geologic formation	ML to OL
Ballasting of pipe during insertion	-None performed-
Pipe installed	12 in. (300 mm) DR 18 FPVC, DIPS (AWWA C900), same lot
Insertion	No pipe rollers used
Recorder	Richard (Bo) Botteicher, UGSI

Construction of Phase II of the Sheridan Reuse Extension Project took place between the Spring and Fall of 2008 and included 24 in. (600 mm) and 12 in. (300 mm) diameter pipe. FPVCP was selected for all of the piping involved with the project.

The east-west interconnect was designed to tie together two separate north-south mainlines in the system, which were also being constructed or slated to be constructed in the future. This cross-connection is composed of 12 in. (300 mm) DR 18 Ductile Iron Pipe Size (DIPS) FPVCP, conforming to the ANSI/AWWA C900-97 standard 21, purple in color, and, in this project, intended for non-potable reuse for irrigation water. The total length of pipe required for the tie-in was ~ 4000 ft (1220 m), and was subdivided into four separate HDD insertions, end to end. The separate drills were 800–1160 ft (244–354 m) in length to account for fusion and lay-down area availability and construction sequencing and methods.

Three of the four project sections were monitored as part of the research initiative. As a result of scheduling and equipment limitations, one of the insertions (Pull 3) was not able to be monitored. The three pulls monitored provided a unique opportunity to correlate the results from three separate yet similar installations. All three installations utilized PVC pipe from the same lot and production run of the extruder, fused by the same technician. Additionally, the same drilling crew and equipment performed each of the three installations. All installations were performed in similar geological formations to the extent that could be determined from project information and prior knowledge. Table 2 presents the parameters that were similar for all three insertions.

Sheridan Reuse Extension Project (Phase II)—Pull No. 1

Pull 1 involved the installation of 1080 ft (330 m) of 12 in. (300 mm) nominal DIPS DR 18 FPVC reuse irrigation water line at a depth of ~8 ft (2.44 m) on average. The actual distance the pipe was pulled was 1120 ft (341 m), as the pipe was pulled in past the insertion pit. The installation took place on March 12, 2008 on the north side and paralleling 144th Avenue. Insertion began at ~9:00 a.m. (MST), and concluded at ~2:00 p.m. (MST). The installation was

located under a sidewalk and curb and gutter section for much of the alignment. Also, the area where the installation was performed was bordering a school property. Densities of drilling fluids collected on site ranged from 8.75 lb/gal (1.05 kg/L) from the mixing tank to 12 lb/gal (1.44 kg/L) from samples of returned drilling fluid. Figure 6 illustrates actual and predicted pull forces based on data from the pull force monitoring equipment and predicted loads.

Sheriden Reuse Extension Project (Phase II)—Pull No. 2

Pull 2 involved the installation of 800 ft (244 m) of 12 in. (300 mm) DIPS DR 18 FPVC reuse irrigation water line at a depth of ~8 ft (2.44 m) on average. The actual distance the pipe was pulled was 885 ft (270 m), as the pipe was pulled in past the insertion pit. The installation took place on March 25, 2008 on the north side and paralleling 144th Avenue. Insertion began at ~8:00 a.m. (MST), and concluded at ~12:00 p.m. (MST). The area where the installation was performed bordered a residential area as well as a small commercial area. There was also an open space available in the vicinity of the installation suitable for pipe fusion and staging. Densities of drilling fluid collected on site ranged from 8.5 lb/gal (1.02 kg/L) from the mixing tank to 13.1 lb/gal (1.57 kg/L) from samples of returned drilling fluid. Figure 7 illustrates actual and predicted pull forces based on data from the pull force monitoring equipment and predicted loads.

Sheriden Reuse Extension Project (Phase II)—Pull No. 4

Pull 4 involved the installation of 1160 ft (354 m) of 12 in. (300 mm) DIPS DR 18 FPVC reuse irrigation water line at a depth of ~9 ft (2.74 m) on average (Fig. 8). The actual distance the pipe was pulled was 1275 ft (389 m), as the pipe was pulled in past the insertion pit. The installation took place on June 19, 2008 on the north side and paralleling 144th Avenue. Insertion began at ~10:00 a.m. (MST) and concluded at ~2:00 p.m. (MST). The area where the installation was performed was bordering a residential area and also a small commercial area. Densities of drilling mud collected on site ranged from 8.4 lb/gal (1.01 kg/L) from the mixing tank to 12.3 lb/gal (1.47 kg/L) from samples of returned drilling fluid. Drilling fluid viscosity information as gathered with Marsh Funnel testing was 62 s for completely mixed fresh drilling fluid prior to circulation into the borehole. Figure 9 illustrates actual and predicted pull forces based on data from the pull force monitoring equipment and predicted loads.

Summary of Installations

The equalization charts in Figs. 10 and 11 illustrate how closely the predicted values from the model compare to the actual values for the installations. Additionally, the graphs reveal whether the predictions fall under or over the actual values attained, thereby indicating whether the model was conservative or under-conservative with its prediction. Points above the trend line indicate a conservative prediction compared to the actual values obtained.

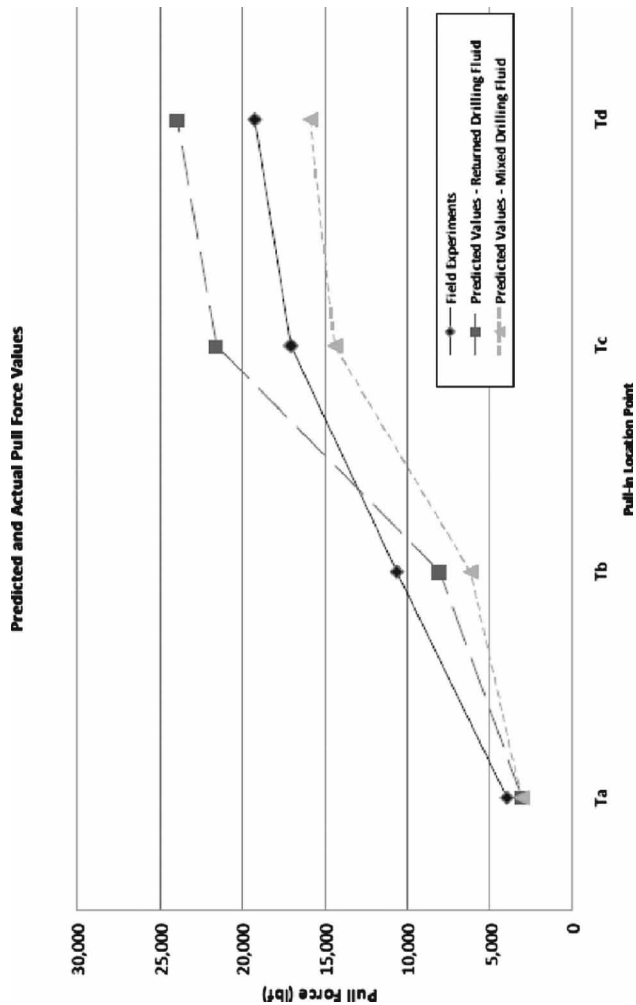


FIG. 6—Actual and predicted pull force values at each of the four critical installation points for Sheridan Reuse Extension Project (Phase II)—Pull No. 1.

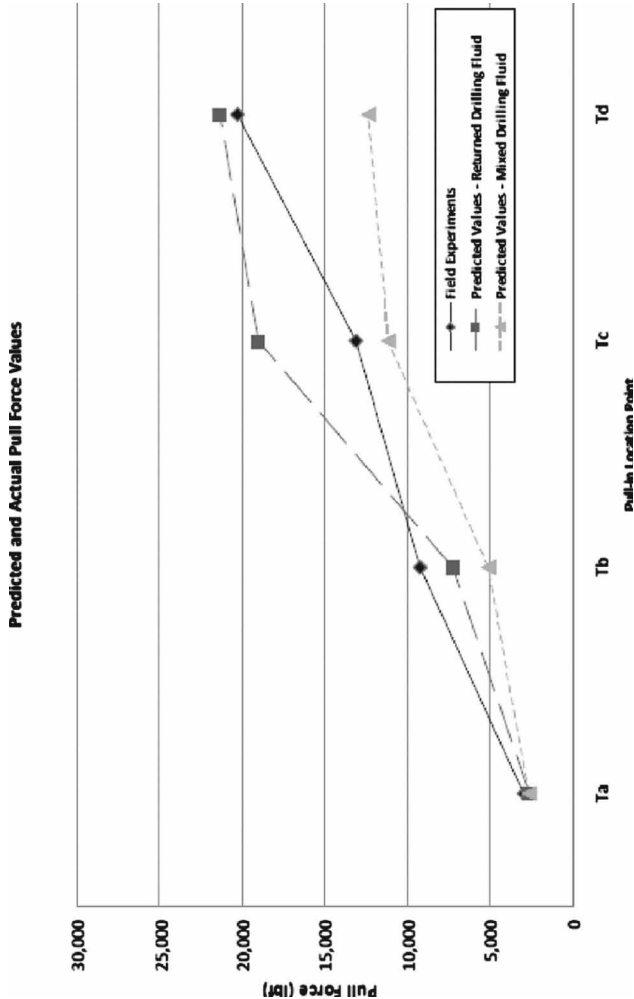


FIG. 7—Actual and predicted pull force values at each of the four critical installation points for Sheridan Reuse Extension Project (Phase II)—Pull No. 2.

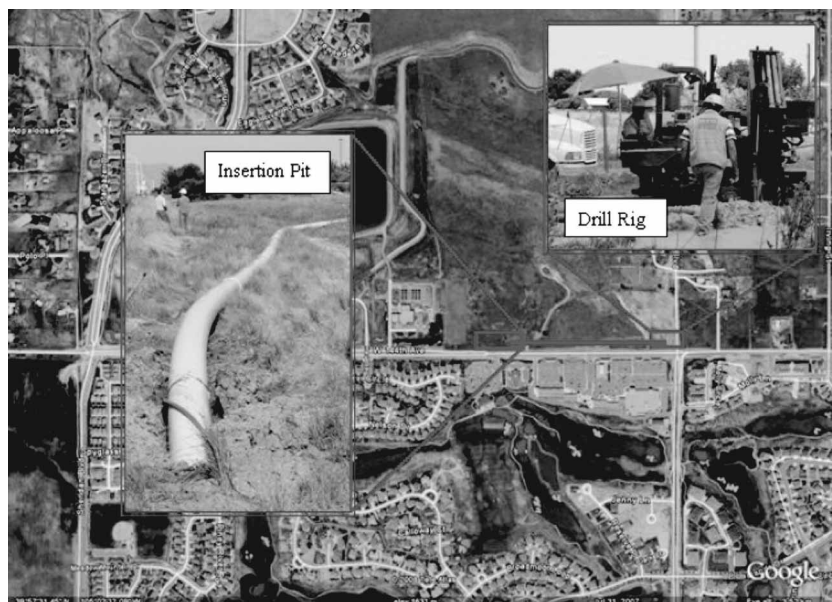


FIG. 8—Location and example photographs of the insertion and drill rig locations for the Sheridan Reuse Extension Project (Phase II)—Pull No. 4.

In Fig. 10, it can be seen that for all three insertions, the higher density values for the drilling fluid produced consistently higher predicted pull force values than those actual values recorded. Two of the three pulls showed close but under-conservative predictions for the first two design points, Ta and Tb. As these points are heavily influenced by the friction of the pipe and ground surface associated with the pull-in and insertion pit, it is not surprising to see these values unaffected by higher density values used for the drilling fluid in the model.

In Fig. 11, it can be seen that for all three insertions, the lower density values for the drilling fluid produced consistently lower predicted pull force values than those actual values recorded. For the same two insertions, the same under-conservative values are again present for the first two design points, Ta and Tb. As was already mentioned, it is not surprising to see these values unaffected by lower density values used for the drilling fluid in the model, much as it was not surprising to see them unaffected by higher density values of the drilling fluid, since the drilling fluid has little impact on these first two design points compared to the role of the surface friction and pipe weight during the first stages of insertion.

Drilling fluid return density varies greatly from project to project. It depends on the way that the borehole has been prepared, the constituency and additives used during the borehole preparation and then during the pipe insertion, the geologic conditions that are being excavated, and other project site and equipment constraints. Therefore, if a reasonable and conservative estimate of maximum pull force is to be accurately made, a reasonable and conservative

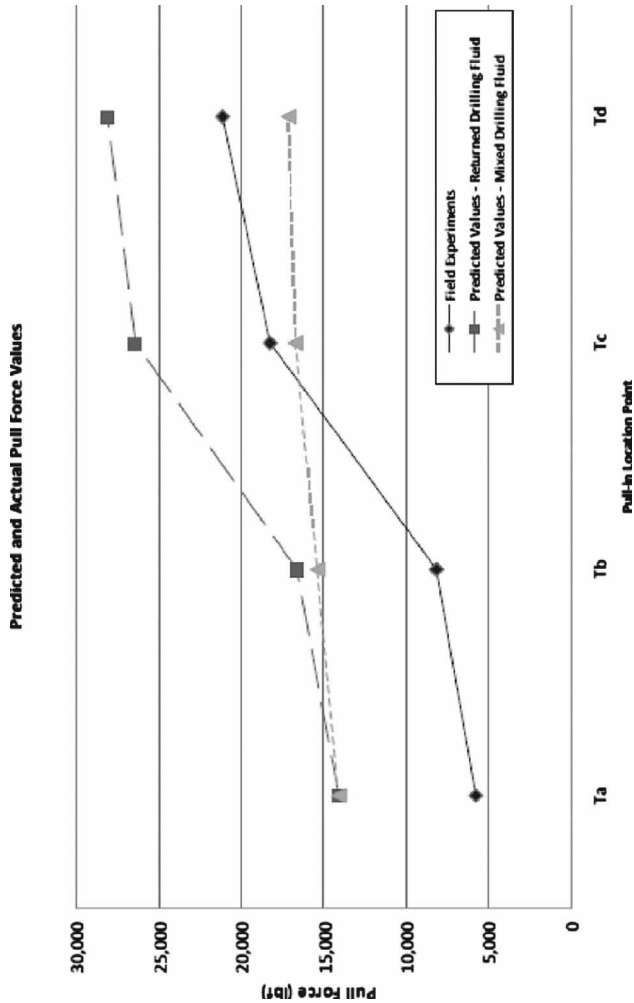


FIG. 9—Actual and predicted pull force values at each of the four critical installation points for Sheriden Reuse Extension Project (Phase II)—Pull No. 4.

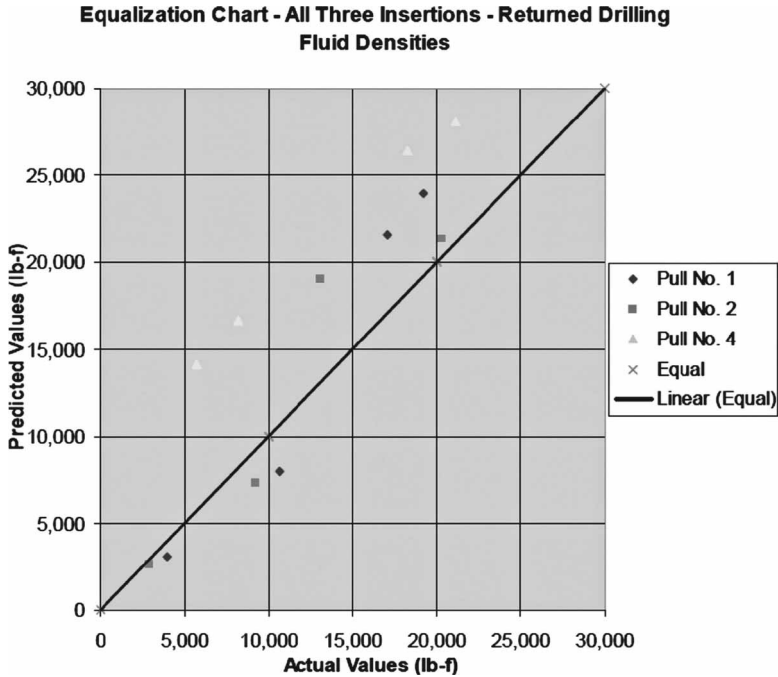


FIG. 10—Equalization chart showing the comparison of the predicted pull force values and the actual values monitored when the drilling fluid densities of the returned fluid values are used.

estimate of the drilling fluid return density is required. Proposed fluid return densities of ~ 12.5 lb/gal [4] are offered as guidance and this case study along with the entire research project found this to be reasonable and applicable to the insertions that were monitored. This particular case study and three insertions show the repeatability of this conservative value creating a conservative maximum pull force estimate, but also demonstrate how the accuracy of such a model is dependent on the appropriate selection of drilling fluid density. The model also illustrates how drilling fluid densities taken from the mixing system will ultimately be under-conservative in prediction and should not be used as an estimate of returned drilling fluid density values for predictive purposes.

Conclusions and Recommendations

A new model, based on ASTM F1962-05, for predicting maximum pull load on FPVCP during HDD installations has been proposed in previous work, and the field validation work has shown it to be acceptable as a predictive tool for pull load determination in such installations.

Three of the eight total field monitored insertions used in the overall research were collected on one project, which provided an opportunity to

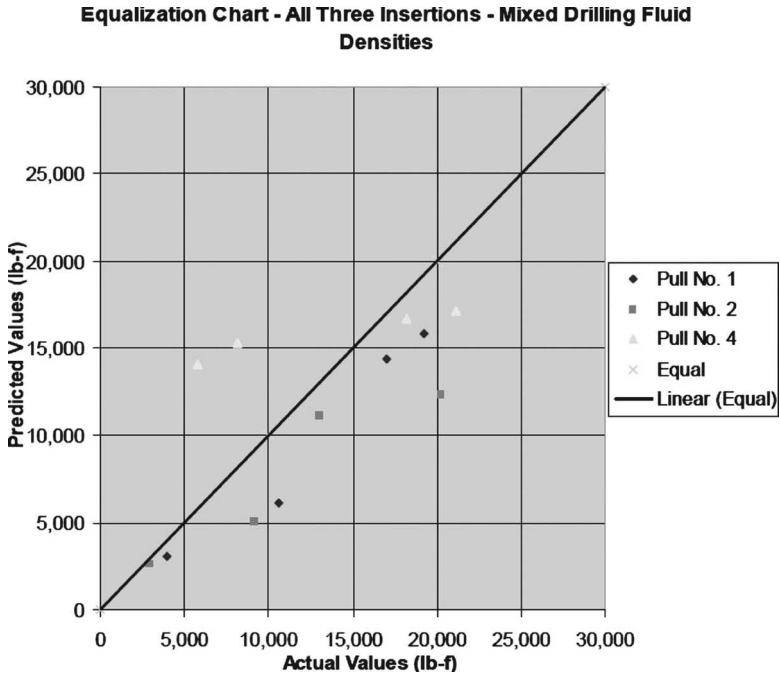


FIG. 11—Equalization chart showing the comparison of the predicted pull force values and the actual values monitored when the drilling fluid densities of the initial mixed fluid values are used.

examine the predictive model over a series of insertions where many of the variables encountered on project to project monitoring basis could be eliminated. The most influential parameter that affects the magnitude of the expected pull force per this model is the buoyancy of the pipe cross-section and the friction that this creates with the bore hole during the insertion process. This component is the single biggest contributor to a maximum expected pull force during an insertion for a typical HDD project involving FPVCP.

The three similar insertions discussed and the manipulation of the drilling fluid density not only show the effect that this component has on the model, but also show how important this parameter is towards the accuracy of any predictive values for the maximum pull force expected. Since successful characterization of the drilling fluid return density is often very difficult to quantify prior to the moment that the pipe is being pulled in, a conservative prediction should be utilized, and moreover, an industry best-practice estimation of 12.5 lb/gal has been shown to provide an accurate yet conservative predictive value based on the research conducted.

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Axial Response of HDPE Pipes as a Result of Installation by Directional Drilling

ABSTRACT: Axial stresses and strains develop in high density polyethylene (HDPE) pipes during installation by directional drilling. This paper examines the engineering design implications regarding these axial stresses resulting from three recent studies. First, the cyclic axial response of the leading end of a HDPE pipe is measured for the cyclic pulling force history experienced as it is pulled into place. Pulling force history measured in the field is approximated and used to measure the response of pipe samples. The study includes evaluation of simple creep modeling to estimate the axial strains, demonstrating that these provide useful, conservative estimates of maximum axial strain. Second, the development of axial tension in the leading section of the HDPE pipe after connection to appurtenances is modeled in the laboratory. It is demonstrated that the conventional 24 h time period over which the pipe is left to recover in length after installation ensures that the axial tensions that redevelop are at low levels. A calculation procedure is introduced to determine the axial force distribution along the pipe during installation and over its service life. Pulling force modeling similar to that outlined in ASTM 1962-05 is extended to consider the effects of pipe and soil deformations, so that cyclic response during installation and axial stress history after installation are determined. The study demonstrates that values of from one quarter to one third of the maximum pulling force can be used to estimate the long term axial stress in the pipe over a 50 year service life (for assessment of potential for slow crack growth). Strain

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measurements in the laboratory simulation of pipe installation and strain calculations for computed response over the service life of the pipe are found to be estimated effectively using time-dependent creep (secant) modulus for the HDPE.

KEYWORDS: design, high density polyethylene, pipe, trenchless installation, directional drilling, axial stress, axial strain, service life

Introduction and Statement of the Problem

Directional drilling involves creation of a curved borehole for the pipe, then installation by pulling the pipe into the borehole as the drill string is pulled back towards the drilling machine [1,2]. This construction technique is well established, and is now commonly used to install steel, high density polyethylene (HDPE), and other pipes under obstructions like rivers, roads, and wetlands, where conventional installation involving trench excavation is undesirable or impossible.

Computational research projects and field studies have explained many of the characteristics of this construction technique, and codes of practice [3] and design manuals [4] are available to guide design of the pipe and the construction operations. However, the time-dependent response of HDPE leads to a number of questions worthy of consideration.

First, Baumert et al. [2] reported on the cyclic pulling loads that develop during installation, as each drill rod is retracted and then removed from the drill string. The effect of these cyclic loads on the tensile axial strains in the HDPE pipe during installation has not been examined, and computational procedures have not been developed for estimating the peak axial strain at the end of pipe installation.

Next, directly after installation the HDPE pipe is given time to contract, allowing recovery of some of the tensile axial strains before the ends of the pipe are fixed to appurtenances. While a common period of length recovery is 24 h [5], the effect of the magnitude of this recovery period has not been examined in detail, nor has the possibility that axial tensions redevelop once the pipe has been fixed (since further length recovery is prevented). While researchers like Lasheen and Polak [6], Baumert et al. [2], and others have developed computational procedures to estimate the maximum pulling force during installation, and therefore estimates of the maximum tensile stress, there has been no calculation procedure developed to quantify what happens after installation, and the long term axial tensions that remain in the pipe.

A multi-component research project has recently been completed by the authors to examine these issues. First, this featured a laboratory study of axial pipe response due to typical pulling force histories [7]. Second, numerical modeling was undertaken of the HDPE pipe both during and after installation, considering the effects of the cyclic loading, the period of pipe contraction after installation, and then the long term pipe response over its service life [8]. While the details of those individual studies have already been reported, the engineering consequences have not yet been discussed, nor recommendations given

regarding methods for undertaking design estimates of short and long term pipe response. The objective of this paper is to provide practical guidance on the consequences of those two investigations, providing answers to the questions raised earlier, and specific recommendations regarding design.

Pulling Force History

Refs [2, 9 and 10 reported on the development and use of a load cell to measure the axial forces applied to the end of the HDPE pipe by the drill string during installation. The end attached to the drill string is the most heavily loaded segment of the HDPE pipe, and will experience the largest tensile axial strains during installation. Figure 1 shows an axial stress history calculated from axial force measured on a 220 mm diameter HDPE pipe. This axial stress history illustrates that stress cycles repeatedly during this process:

- (a) Axial stress peaked as the new pipe moved forward during retrieval of each segment of the drill string (as each rod was pulled up into the drilling machine and a similar length of the HDPE pipe was pulled down into the borehole); and
- (b) Axial stress dropped to a minimum as the forward motion stopped temporarily, so the specific drill rod could be removed from the drill string, and the carriage advanced and refastened to the remaining length of the drill string.

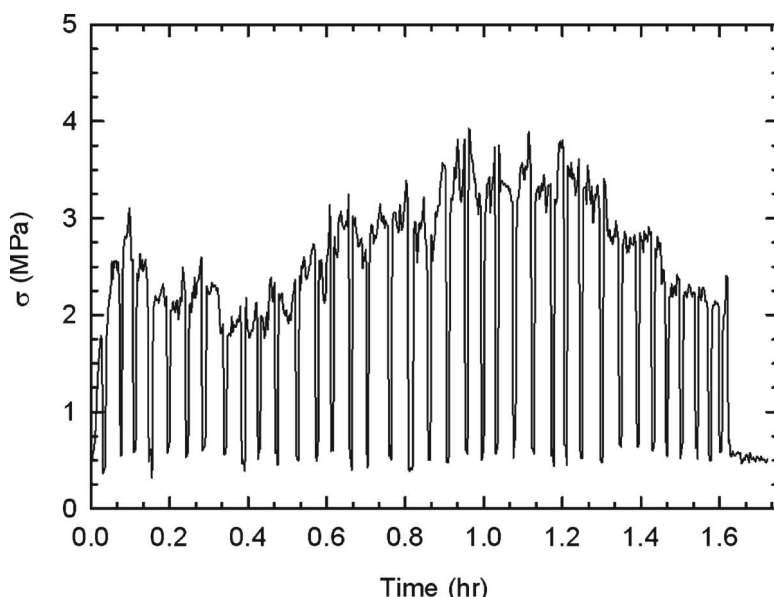


FIG. 1—Axial stress history calculated by Cholewa [11] from measured pulling forces reported by Baumert [9].

The cyclic stresses continued as these two operations were repeated until each of the drill rods was recovered, and the pipe was pulled all the way into the borehole. The axial stress history shown in Fig. 1 features recovery of 39 drill rods, and the associated load cycles. Each cycle took approximately 2 min to pull a drill rod back up into the drilling rig, and about 30 s to remove that drill rod and prepare to begin recovery of the next rod.

Questions that arise from this cyclic loading history include:

- (a) What is the effect of load-unload cycles on the axial strains that develop in the leading section of the HDPE pipe attached to the drill string?
- (b) Is there a practical way of estimating the maximum axial strains based on the peak and residual pulling forces?

Other questions arising from consideration of the pipe installation process include:

(c) How long should the pipe be allowed to recover its length before it is attached to appurtenances (the common convention is to leave the pipe to recover over 24 h)?

(d) What are the long term axial stresses in the pipe and where are they greatest?

(e) How do these long term stresses relate to the axial stress associated with the maximum pulling force?

Laboratory Measurements of Axial Pipe Response

A laboratory study was performed to examine the response of the leading end of a sample of HDPE pipe with diameter of 150 mm, dimension ratio (DR) of 17 (the ratio of external diameter to pipe thickness) during and after pipe installation. First, the measured pulling forces were used as a guide to determine the axial stress history to be applied, as discussed by Cholewa [11]. Next, the complex pulling force history was approximated as a series of uniform stress cycles, between maximum and minimum axial stresses of 3.8 and 0.58 MPa respectively for the DR17 product, Fig. 2. This had peaks sustained for 2 min intervals (corresponding to periods where the pipe is being pulled into place), and troughs sustained for 30 s (when the HDPE pipe is not advancing and a rod is being removed). Samples like that shown in Fig. 3 were prepared, where the test pipe was fitted with stub-ends and restrained with flange connections. Placing the sample in a universal test machine, the axial response (length change Δ) was studied under applied axial force history P . Furthermore, the fixture was fitted with a stiff internal rod and load cell for use during the last stage of the experiment. To model the "fixed length" condition after the pipe is fixed to appurtenances, the rod is engaged with the two ends of the sample at fixed length, and the load cell is used to measure axial forces in the restraining rod. The uniaxial loading conditions produced by this loading system represent a stress state that approximates that in the pipe in the field, where radial and hoop stresses are small compared with the axial pulling stresses.

Figure 4 shows the imposed or measured strain and stress histories for one such test. This test featured 25 load cycles during the "simulated installation," a

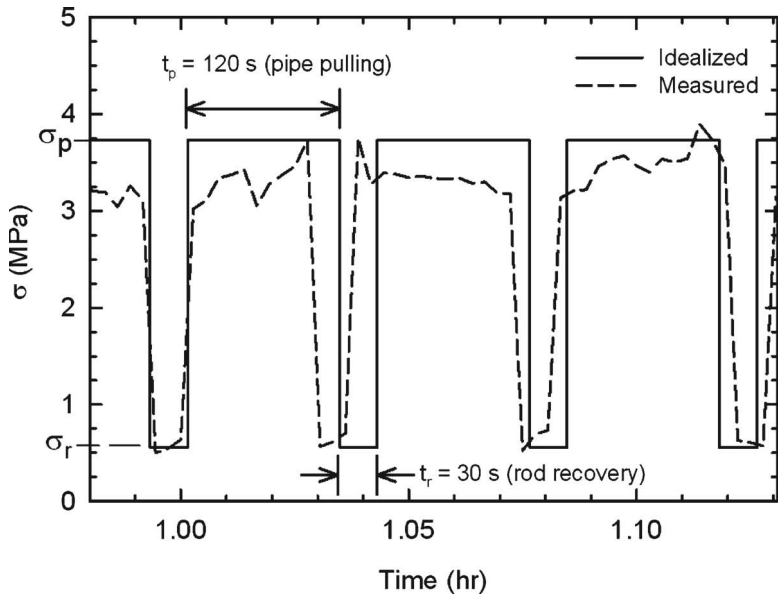


FIG. 2—Idealized and calculated stress histories from measured pulling forces reported by Baumert [9] over a 0.15 h period [11].

1 h period of length “recovery,” and then 100 h with axial “restraint,” where the total axial force that redevelops in the pipe is recorded using the internal load cell.

These tests to evaluate the response of the leading segment of pipe provide reasonable assessment of axial deformations during cyclic loading and the free length recovery phases. However, during the final fixed length testing, the assumption is that this leading segment of pipe has the fixed length condition

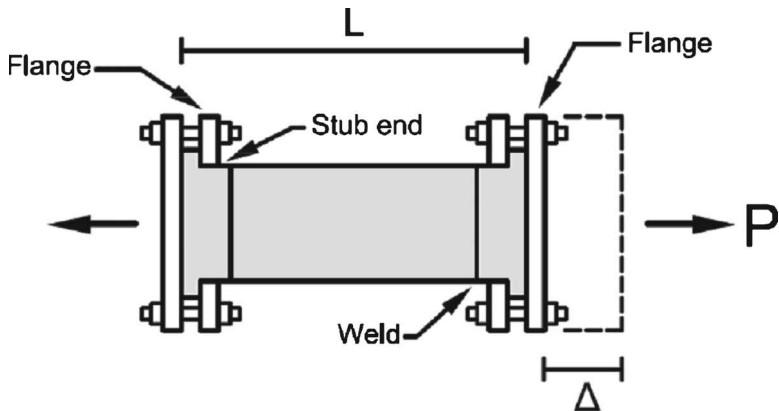


FIG. 3—Sample for tests on HDPE pipe [7,11].

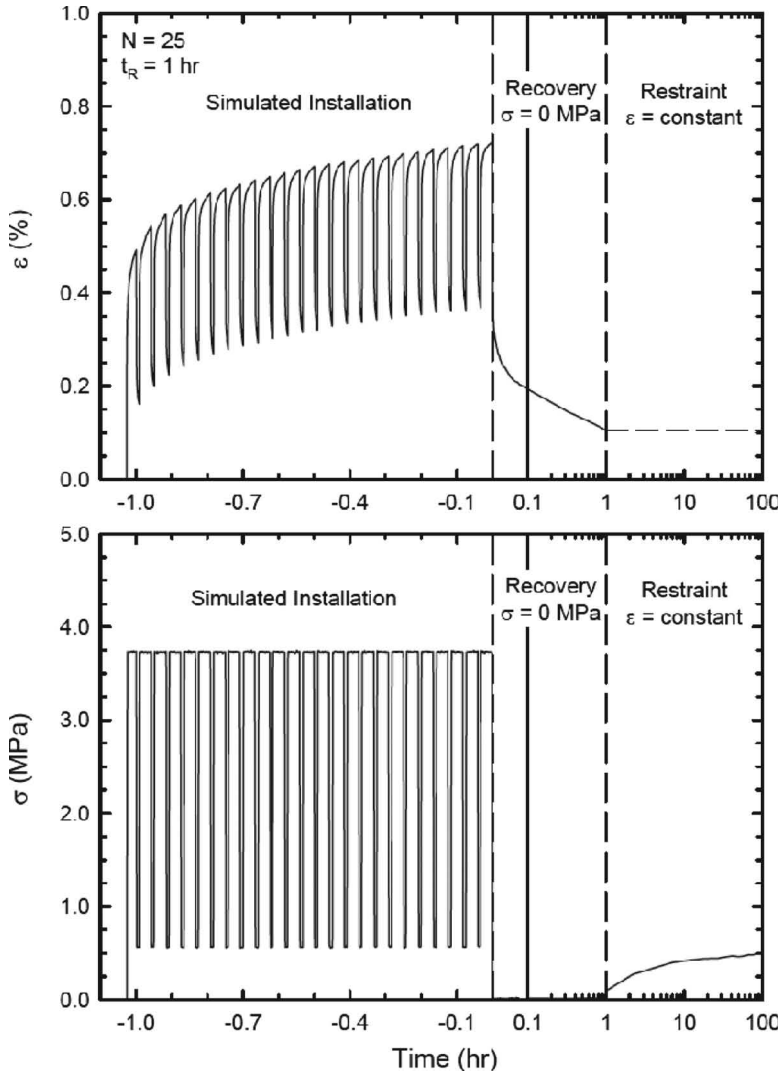


FIG. 4—Stress and strain history of HDPE pipe test [7,11].

corresponding to the average for the pipe after it is installed and fixed to appurtenances. The validity of this assumption is evaluated in a later section, where the full pipe length is modeled during and after installation.

Since the effect of pressure differential across the reamer is not clear, Cholewa et al. [7] conducted tests for two limiting cases. “Upper bound” pipe response was represented by a series of uniform magnitude load cycles, each reaching the maximum axial stress of 3.8 MPa described earlier. The procedure for calculating pulling force in ASTM F1962 [3] and other evidence indicate that pulling force increases approximately linearly with pipe installation length.

TABLE 1—Comparison of measurements and calculations for strain at the end of installation (pipe pullback).

Approach	Peak strain during test (%)
Test [7]	0.72 ($\times 0.85$ if linearly increased)
LVE (Ref 7 based on model of Ref 12)	0.68
NVE (Ref 7 based on model of Ref 13)	0.76
$E_{\text{secant}} = 540$ MPa (based on peak pulling stress over the time of installation, and secant modulus obtained from data of Ref 12)	0.69

Therefore, “lower bound” pipe response was obtained by steadily increasing the maximum axial tension during each rod recovery cycle from the minimum of 0.58 MPa to the maximum of 3.8MPa.

Table 1 contains values of maximum axial strain obtained at the end of the upper bound simulation of pullback, a value of 0.72 %. Strain at the end of the lower bound simulation was 85 % of the final upper bound strain. Table 2 contains values of axial tension redevelopment in the test pipes, for three different time periods during which the pipe is left free to recover viscoelastic strains: 2 minutes, 1 hour, 24 hours. In each case, the axial tension recovered is that measured after 100 hours of axial restraint.

Calculations of Axial Response of Leading Edge of the Pipe

Use of viscoelastic modeling was then examined to determine axial strain in the leading edge of the pipe during installation (the most heavily loaded section that is expected to have peak axial installation strain). Calculations of axial strain in the pipe were then undertaken using two viscoelastic models:

- LVE: The linear viscoelastic model for HDPE; and
- NVE: The nonlinear viscoelastic model for HDPE of Ref [13].

These calculations were performed using the finite element analysis AFENA (a modified version of the program developed by Carter and Balaam [14]).

Figure 5 shows the strains calculated over the course of the laboratory test, using these two computational models for HDPE and considering the pulling force history that was imposed. Figure 5 also shows the history of average strain during the recovery phase of the tests, when the axial force is zero and the

TABLE 2—Axial stress redeveloped after 100 h for different periods of pipe contraction (length recovery after installation before fixing pipe ends to appurtenances).

Recovery Period	Axial Stress when Fixed for 100 h (MPa)
24 h	0–0.1
1 h	0.42–0.53
2 min	0.64–0.87

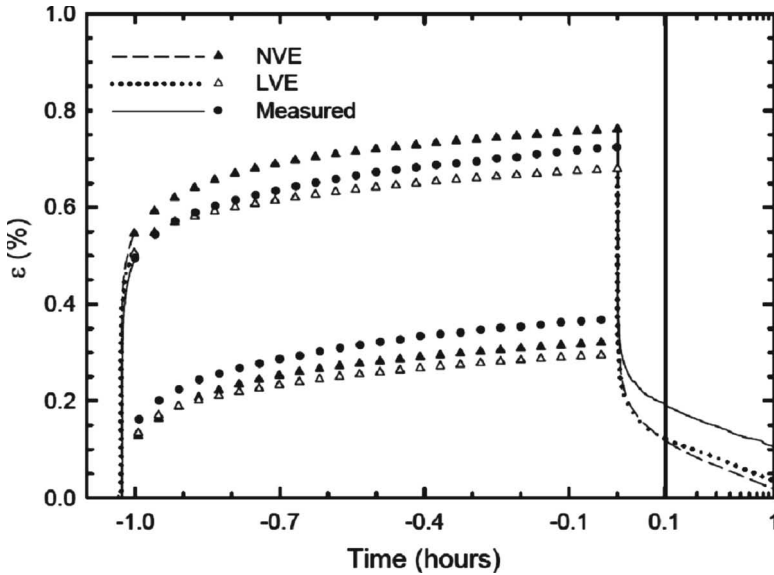


FIG. 5—Axial strain during and after loading—measurements and calculations [7,11].

sample is free to contract axially. Average strain measured in the sample is also shown in this figure. Table 1 contains values of the axial strain measurements described in the previous section, and these two calculations for the end of pipe installation (pullback), strain values of 0.68 and 0.76 % for the LVE and NVE models, respectively.

The peak values of measured strain at the end of each “maximum load” (rod retraction) part of the load cycle lie between the linear and nonlinear visco-elastic solutions. Both the LVE and NVE solutions, however, overestimate the amount of strain recovered during the “minimum” load (rod removal) part of the load cycle, likely because the HDPE material response also includes viscoplastic strains (time-dependent strains in the material that are permanent).

A simple design calculation of the peak axial strain on the leading end of the pipe might be made assuming a “creep” condition throughout installation (constant axial stress of 3.8 MPa). Table 1 contains this simple estimate, based on a secant modulus for HDPE of 540 MPa corresponding to time of 70 min (interpreted from data presented by Moore and Hu [12]). The strain that results (0.69 %) is similar to those obtained by the other calculation procedures. It appears, then, that simple design estimates of maximum axial strain at the end of pullback can be obtained using maximum axial pulling stress and secant modulus for the total time taken for the pullback operation.

Calculations of Complete Pipe—Installation and Post-Installation Behavior

Refs [15 and 16 reported on new models developed to characterize the time-dependent behavior of HDPE for cyclic loading histories, and those involving

strain reversal. Ref [8 describes the implementation of pulling force calculations extending the procedures outlined in ASTM F1962 [3], to represent the full loading history during and after installation. Explicit modeling of each load cycle during pullback includes the forward motion of the leading end of the pipe as one drill rod is pulled into the drill rig, and the time period during which there is zero forward motion so the drill rod can be removed from the drill string, and so the rig can retract and be re-attached to the next drill rod to be recovered. A period of length recovery is modeled after installation—where parts of the HDPE pipe are experiencing strain reversal, but where other parts are still under axial tension as a result of soil-pipe shear stresses (segments that continue to extend under the applied tensions). The procedure then computes subsequent response “in-service” when the total length of the pipe is fixed. However, parts of the pipe extend under sustained axial stress (those in the middle of the installed pipe), and others parts contract (at the pipe ends).

To illustrate this process, results are summarized here for the example installation shown in Fig. 6. The installation being modeled featured a pipe with total length of 220 m, 750 mm outer diameter pipe, with wall thickness of 40 mm (DR of 19). Load cycles featuring recovery of rods of length 9.5 m over a 22 s interval, followed by 20 s of zero movement, producing cycles of loading and unloading during installation. Coefficients of friction of the pipe with ground surface and borehole were set at 0.3 and 0.5, respectively, and adhesion between the pipe and the soil was neglected. Soil-pipe interaction with shear stiffness of 1 MN/m per meter of borehole length was used (see Ref [8 for further discussion of this interaction stiffness). Drilling mud with specific gravity of 1.5 (a conservative value representing low levels of cuttings removal from the drilling mud) and dynamic viscosity of 1.0 Pa s (10 Poise) was modeled as continuously pumped during pullback at a rate of 10 L/s. The choices are not the only possible values, but the overall findings of the study in relation to short versus long term pipe response do not change if other choices are made.

The linear viscoelastic-viscoplastic constitutive model described by Ref [16 was used to model the mechanical response of the pipe during and after installation, a model that can capture tensile and compressive strain and strain

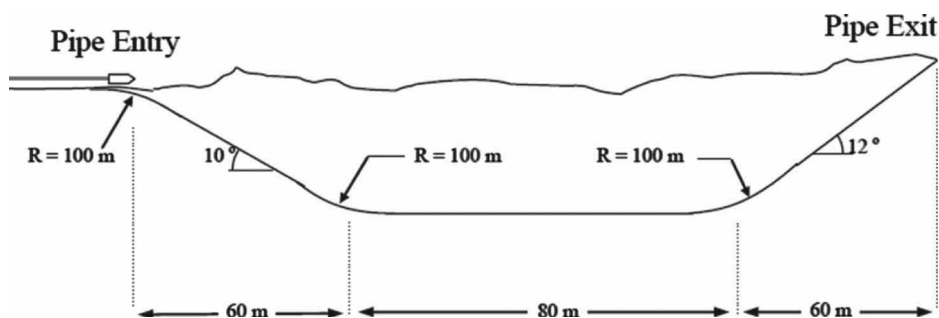


FIG. 6—Example HDPE pipe installation [15].

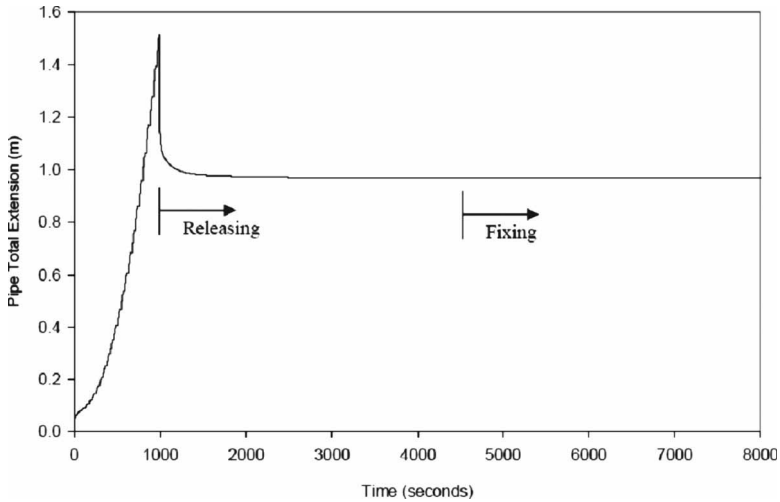


FIG. 7—Calculated extension during HDPE pipe installation [15].

reversal. Parameters were those for HDPE at 23°C; the effect of higher or lower temperature might be considered in a subsequent study.

To account for elastic and viscoelastic recovery, an extra 5 m (representing about 2.4 % of the total pipe length) was pulled out of the pipe exit point. Post-installation modelling featured just 1 h of strain recovery before fixing both ends of the pipe to existing infrastructure (so as to obtain a conservative estimate of tension recovery at the leading end of the pipe).

Figure 7 shows the change in pipe length calculated during and after pipe installation. This indicates that total length increase for this project was calculated to be over 1.5 m just before the end of the pipe was released. This reduced to about 1.0 m during the 1 h period of length recovery.

Figure 8 shows the distributions of axial stress along the pipeline at various points of the construction process:

- (1) At the end of installation just before the leading end of the pipe was released;
- (2) Immediately after release of the end of the leading end of the pipe;
- (3) 1 h later when the ends of the pipe were fixed against axial movement;
- (4) 10 h after fixing;
- (5) 1 month after fixing; and
- (6) 3 years after fixing.

During installation the maximum axial tension of 7.4 MPa was calculated to develop at the leading end of the pipe (220 m from the tail). On release, the axial stress on that leading end decreased to zero, and an approximately triangular distribution was calculated, with peak tension of about 4.1 MPa near the pipe mid-point. Over time, this maximum value of tension decreased as recorded in Table 3, dropping below 2.0 MPa before the end of 1 month (30 days). Subsequent stress reductions were modest, and long term axial

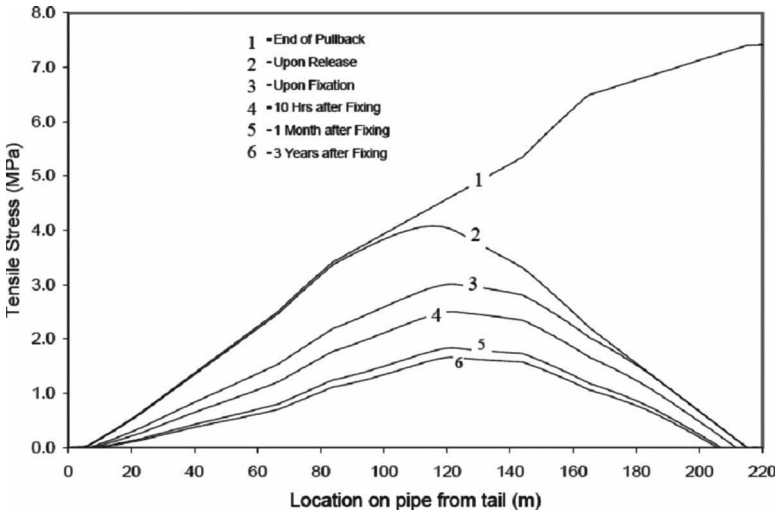


FIG. 8—Axial stress distributions after HDPE pipe installation [15].

stress remained just below 25 % of the short term value. Calculations for other configurations are included in Chehab (2008), and these indicate that long term axial stress is generally between one-third and one-quarter of the maximum axial stress that develops during installation.

The calculation procedure also provides the complete pattern of axial strain along the pipe, both during and after installation. Chehab (2008) indicates that after 3 years, peak tensile strain for this installation reaches 0.8 % (Table 4). If the long term axial stress of 1.9 MPa is used with a ‘creep’ modulus for HDPE at 3 years, 240 MPa [12], axial strain of 0.79 % is calculated. This indicates that very reasonable “engineering” estimates of creep strain can be made using secant creep modulus for HDPE together with an estimate of the long term, maximum axial stress, a much more straightforward calculation compared to the viscoelastic-viscoplastic soil-pipe interaction modeling of Ref [15]. That more sophisticated calculation procedure might be reserved for high cost, high risk projects.

TABLE 3—Peak stress in example pipe at different times.

Time	Peak Stress
End of installation	7.4 MPa (100 %)
Immediately after release	4.1 MPa (55 %)
When fixed	3.0 MPa (40 %)
1 month later	1.9 MPa (25 %)
3 years later	1.7 MPa (23 %)

TABLE 4—*Peak strain obtained using two different calculation procedures.*

Time	Peak strain after 3 years
Chehab [15] procedure	0.8 %
$E_{\text{secant}} = 240 \text{ MPa}$ (modulus of Ref 12)	1.9 MPa (25 % of peak) produces 0.79 %

Conclusions and Recommendations Regarding Design Calculations

Field and laboratory measurements, as well as computer modeling have been used to examine the effect of cyclic pulling force histories on HDPE pipe pulled into place using directional drilling. The conclusions are as follows:

- (1) Cyclic stress history results in steady increase in tensile axial strain at the end of each pulling cycle.
- (2) Reasonable estimates of these tensile axial strains can be obtained by calculating creep under a constant axial pulling force; linear viscoelastic modeling underestimates that peak axial strain, while nonlinear viscoelasticity overestimates that peak (the measurements fall in between); both linear and nonlinear viscoelastic models overestimate the strain recovery—measurements indicate that permanent (i.e., viscoplastic) strains remain which are significantly higher than the viscoelastic (linear or nonlinear) calculations.
- (3) The ability of the HDPE pipe to survive those installation forces could be made by comparing that strain value with the short term (e.g., 24 h) strain capacity of the material.
- (4) Axial tensions redeveloped in the test specimens; however, these are lower than the long term axial stress capacity for slow crack growth; further, the standard wait time of 24 h reduces the axial tensions that occur subsequently to very low values; furthermore, calculations for the entire pipe indicate that these end sections actually go into compression, since the soil-pipe shear stresses leave the central part of the pipe in tension, and this central section continues to experience increasing tensile axial strain; the ends continue to contract therefore (so that total pipe length is fixed), so these ends then develop compressive axial stress; as a result the critical section of the pipe after installation is the central part (provided the leading end survived installation, it should continue to be fine).
- (5) Axial tension in the center of the pipe immediately after the end is released by the HDD machine is about half the peak value that developed during installation; this value continues to decrease with time, dropping in this example to about 1/3 of the peak after it is fixed, and down to 1/4 or less of the peak after 3 years. Conservative estimates of service life could be made considering, say, 1/3 of the peak axial pulling stress (ignoring, of course, any bending stresses in the pipe) i.e., determining the time to fracture under that stress if it were sustained. Reasonable estimates of the axial strain can be made using creep

models (or a long term HDPE modulus) with perhaps 1/3 of the peak axial tension, over the defined service life (e.g., 50 years).

6. For the cases considered here, the use of current practice restricting short term axial stresses in HDPE pipes to 9 MPa should ensure that long term axial stresses remain below the long term allowable limits.

These studies were conducted neglecting other potential sources of axial stress in the pipe. For example, it is assumed that the temperature does not change after the pipe is fixed at its ends (i.e., it is assumed that the pipe reaches ground temperature before the end of installation). Stresses resulting from subsequent temperature changes (say due to transport of fluids with different temperatures) would need to be added to the axial stresses from pulled in place installation reported here, as would axial stress resulting from internal fluid pressures during service (the high Poisson's ratio of HDPE means that axial stress increases by almost half of the hoop stresses that result from internal pressure).

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