Culvert Rehabilitation to Maximize Service Life while Minimizing Direct Costs and Traffic Disruption

Literature Review

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ABBREVIATIONS

AMS	Asset management system
CASP	Corrugated aluminized steel pipe
CCFRPM	Centrifugally cast fiberglass reinforced polymer mortar
CCGRP	Centrifugally cast glassfiber reinforced plastics
CFP	Collated fibrillated polypropylene
CIP	Cured-in-place
CLCMP	Concrete lined corrugated metal pipe
CM	Corrugated metal
CMB	Corrugated metal box
CMP	Corrugated metal pipe
CSCP	Corrugated steel culvert pipe
CSM	Chopped strand mat
CSP	Corrugated steel pipe
DOT	Department of transportation
GIP	Grout-in-place
GRFC	Glass fiber reinforced concrete
GFRS	Glass fiber reinforced shotcrete
GRP	Glass fiber reinforced polyester
FRC	Fiberglass reinforced cement
FRP	Fiberglass reinforced plastic
FRPC	Fiber reinforced polymer composite
FRS	Fiber-reinforced shotcrete
HDPE	High density polyethylene
LRFD	Load and resistance factors design
NDI	Non-destructive inspection
NDT	Non-destructive testing
PCC	Portland cement concrete
PE	Polyethylene
PP	Polypropylene
PS	Pipe stiffness
PU	Polyurethane
PVC	Polyvinyl chloride
RCP	Reinforced concrete pipe
SDR	Standard dimensional ratio
SFRS	Steel fibre reinforced shotcrete
SL	Segmental lining
SSPP	Structural steel plate pipe
RTRP	Reinforced thermosetting resin plastic
VCP	Vitrified clay pipe
UCP	Unreinforced concrete pipe

1. INTRODUCTION

1.1. DEFINITION OF CULVERT

Traditional definitions of culverts are based on the span length rather than pipe function or structure type. For instance, FHWA (1970) states that “structures over 20 ft in span parallel to the roadway are usually called bridges and structures less than 20 ft in span are called culverts even though they support traffic load directly”.

Arnoult (1986) describes a culvert as a drainage opening beneath embankment, usually a pipe, designed to flow according to open channel equation.

AASHTO (1991) brings an expanded definition of a culvert: (1) a structure usually designed hydraulically to take advantage of submergence to increase hydraulic capacity; (2) a structure used to convey surface runoff through embankment; (3) a structure, as distinguished from bridges, that is usually covered with embankment and is composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert; and (4) a structure that is 20 ft or less in centerline length between extreme ends of openings for multiple.

Culverts are pipes typically located under roadways, embankments, or service areas that allow passage of stormwater, and are built with straight horizontal alignment and single grade (vertical alignment). Thus, culverts are different from storm sewers, which may contain catch basins or drop inlets for collecting water and are generally much longer. Although the length of culverts is not limited, most existing culverts are rather short (e.g., many culverts are located under two-lane roadways and are up to 50 ft long).

1.2. TYPES OF CULVERTS

Two basic types of culverts can be distinguished with respect to load carrying behavior: rigid and flexible (Figure 1). Rigid culverts (e.g., concrete culverts, either non-reinforced or reinforced) experience high bending moments and little deformation. They benefit from good soil support through less nonuniformity in earth pressures around the outside of the pipe, since this reduces the bending moments that develop. When vertical loads are applied, zones of tension and compression are created in the pipe wall. Shear stress in the haunch area can be critical for heavily loaded rigid pipe on hard foundations, especially if the haunch support is inadequate. Flexible culverts (e.g., metal or thermoplastic culverts) experience little bending moment but significant deformations. Their ability to resist deformation is a result of the composite action of the culvert barrel and the surrounding soil. The deformations in flexible culverts act to change the magnitude of the earth pressures that are acting (in contrast to rigid culverts that have no significant change in earth pressures as a result of pipe deformation). Except in exceptional cases, the vertical diameter in flexible culverts decreases as a result of soil loading, and most flexible culverts experience horizontal diameter increases (thermoplastic pipes of low hoop stiffness placed in very stiff backfill can experience circumferential shortening sufficient to generate horizontal diameter decrease).

Culverts may have different shape (e.g., circular, arch, elliptical, box, etc) and be made with single or multiple barrels. The most common culvert materials are concrete (reinforced or non-reinforced), corrugated metal (aluminum or steel), and polymer (PVC or PE), either profiled (e.g., corrugated, ribbed) or plain. Standard shapes of concrete and metal culverts are shown in Table 1. Other materials that have been used for culverts are masonry, wood, vitrified clay pipe, fiber reinforced cement, fiberglass, and cast iron.

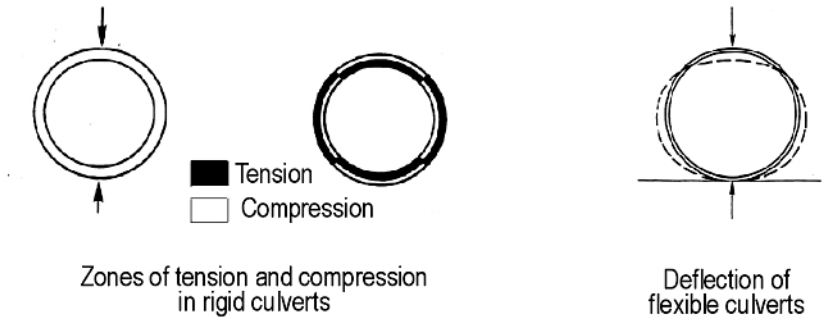
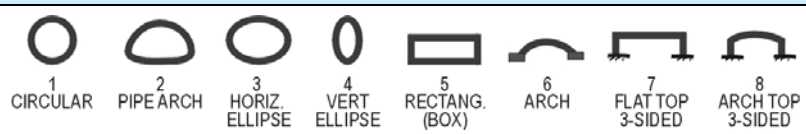
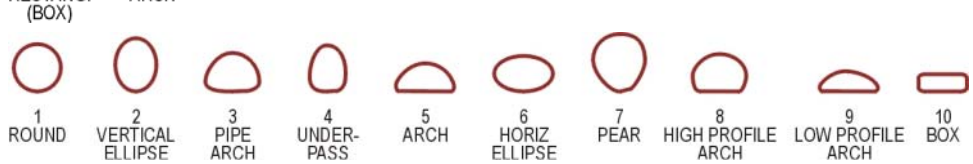
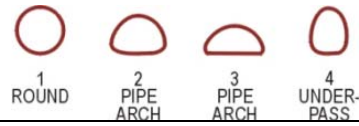


Figure 1. Load carrying behavior of rigid and flexible culverts (after Ballinger and Drake, 1995)

Culverts can be made from prefabricated sections that are factory produced (precast concrete pipes, corrugated metal pipes, or plastic pipes) or assembled in the field (cast-in-place reinforced concrete, corrugated metal structural plates). Reinforced concrete culverts are generally precast prior to delivery to the project site. If cast-in-place, they are typically single or multi-cell box culverts. Corrugated metal structural plate culverts are made from individual plates that are formed, curved, and galvanized at the factory, and bolted together at the job site to form a full circular or noncircular pipe, arch, or pipe-arch structure.

Table 1. Standard shapes of concrete and metal culverts (based on Ballinger and Drake, 1995)

Material	Standard shapes
Concrete, precast (Reinforced usually)	
Concrete, cast-in-place (Reinforced)	
CMP, steel	
CMP, aluminum	
CM structural plate, steel or aluminum	

To extend service life, steel used for culverts are typically galvanized. Various protective coatings and pavings may be applied to add corrosion protection (the coatings are applied to cover the entire periphery of the pipe) or abrasion protection (a sacrificial layer of resistant material is applied in the invert of the pipe). Extra metal thickness is considered only after protective coatings and pavings have been considered (Caltrans, 2006). Aluminum may display inferior abrasion characteristics compared with steel in non-corrosive environments, however, Aluminized Steel (Type 2) can be considered equivalent to galvanized steel for abrasion resistance.

Although thermoplastic pipes can be made with solid walls (i.e., have a continuous wall of uniform thickness), they are often manufactured with profiled walls to maximize the flexural stiffness of the pipe for a given amount of polymer. Corrugated HDPE pipes fall into these three categories (Figure 2): (1)

type C have a corrugated surface both inside and outside, (2) type S have a smooth surface inside (a liner) and a corrugated surface outside, and, (3) type S honeycomb (also referred to as type D in some of AASHTO specifications) have essentially smooth inner wall joined to an essentially smooth outer wall with annular or spiral connecting elements. Both type C and type S pipes can have either annular or helical corrugations. PVC pipes with profile walls fall into one of three categories (Figure 3): (1) open profile pipes have rib enforcements exposed on the outside of the pipe; (2) closed profile pipes are often described as an I-beam or honeycomb; and, (3) dual wall corrugated pipes feature a smooth-wall waterway braced circumferentially with an external corrugated wall.

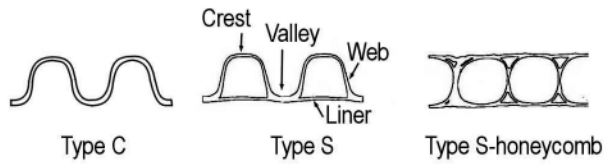


Figure 2. Profiled walls of HDPE pipe

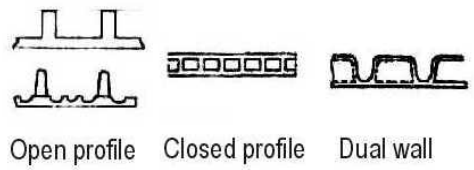


Figure 3. Profiled walls of PVC pipes

1.3. NUMBER OF EXISTING CULVERTS IN U.S.

The Status of the Nation's Highways, Bridges, and Transit: Conditions and Performance (FHWA, 2004) reported a total of 118,394 culverts in the bridge inventory in U.S. This count refers to structures with no deck, superstructure, nor substructure, but rather self-contained units under roadway, and typically constructed of concrete or corrugated steel.

The total number of culverts in U.S. is much larger, however the number is not known at this time. Various DOTs, local agencies and other government entities, e.g., United States Department of Agriculture Forest Service (USDAFS), have been actively working on culvert inventories in order to facilitate their asset management. Some examples are as follows:

- The Oregon DOT started the field inventory of culverts in July 2007, and, as of January 2009, has inventoried more than 4,000 culverts throughout Oregon with many culverts yet to be entered into the inventory (ORDOT, 2009).

- The Ohio DOT has developed the Culvert Management Manual (ODOT, 2003) to aid in the inventory and inspection of culverts under highways maintained by the state.

- The Washington State DOT completed a state wide inventory of its highway system in 2007 and inventoried a total of 6,469 culverts statewide (Wilder and Barber, 2008). Within this number, a total of 1,431 culverts were inventoried in Jefferson County, WA (Till et al., 2000).

- The Vermont Agency of Transportation (VTrans) has been working on an inventory and condition assessments of all state owned small culverts (i.e., culverts less than 72 in. in diameter). The 2007 field data show 2,739 culverts, which covers approximately 65% of the total highway mileage inspection goals (VTrans, 2008)

- In Alaska, approx 900 stream crossing culverts have been inventoried over the last six years and it is estimated that at least an additional 5,000 or more stream crossings have not yet been identified (ADF&G, 2009)

2. CONDITION ASSESSMENT OF CULVERTS

2.1. TYPES OF CULVERT DETERIORATION

2.1.1. NOTABLE REFERENCES

This section reviews types of culvert deterioration that can be found in concrete, metal and thermoplastic culverts, which are relevant for culvert condition assessment. Several FHWA's manuals have been identified to contain comprehensive information on the topic in concrete and corrugated metal culverts:

- ***Culvert Inspection Manual***, Supplement to the Bridge Inspector's Training Manual (Arnoult, 1986), offers a detailed description of defects in concrete and corrugated metal culverts, explaining the appearance of defects and causes that lead to their development, and giving guidelines on how to conduct and document culvert inspections.
- ***Culvert Repair Practices Manual*** (Ballinger and Drake, 1995) presents specific signs of distress in culverts with different barrel materials (Table 2) and shapes, which is partially based on the information presented in the Culvert Inspection Manual.
- ***Bridge Inspector's Reference Manual (BIRM)*** (FHWA, 2006), a revised version of a BIRM 2000 manual, adds a review of advanced inspection techniques for inspection of steel and concrete bridges that can be used for inspection of steel and concrete pipe culverts.

Table 2. Defects in concrete and corrugate metal culverts, as listed in Ballinger and Drake (1995)

CONCRETE CULVERTS		CORRUGATED METAL CULVERTS	
Cast in place culverts	Precast culverts	CM pipe culverts	CM structural plate culverts
Cracks, spalls	Misalignment	Shape distortion	Shape distortion
Undermining	Joint defects	Misalignment	Misalignment
Durability problems	Longitudinal cracks	Joint defects	Joint defects
	Transverse cracks	Dents and localized damage	Seam defects
	Spalls	Durability problems	Circumferential seams
	Slabbing		Dents and localized damage
	Durability problems		Durability problems
	End section drop-off		Footing defects

The following useful sources of information related to thermoplastic culverts have been identified:

- A study conducted for PennDOT (Selig, 1995) provided an understanding about how HDPE pipes perform and should be specified.

- NCHRP Report 429 (Hsuan and McGrath, 1999) investigated field retrieved samples from cracked HDPE culvert pipes throughout the US (obtained from 29 field sites that had been identified from a questionnaire sent out to 50 state DOTs, federal and local agencies and engineering consultants), and conducted laboratory testing of commercially available new pipes (14 samples). The study provided understanding about failure mechanisms in HDPE corrugated pipes.

- A study conducted for SC DOT (Gassman et al., 2000) evaluated HDPE pipes throughout South Carolina (45 HDPE pipes were selected statistically based on geographical location, diameter, use and age) and documented observed defects.

- NCHRP Report 438 (McGrath and Sagan, 2000) investigated local bucking behavior of profile wall

thermoplastic pipes. The study conducted a literature review and a laboratory testing. A design model was developed to predict the capability of the pipe in compression as governed by local buckling.

- A report prepared for FL DOT (Hsuan and McGrath, 2005) analyzed pipe stresses in buried corrugated HDPE drain pipes and, while developing specification requirements for long-term service life of corrugated HDPE pipes, elaborated on stress crack resistance of corrugated HDPE pipes, antioxidant formulation in the pipe to ensure oxidation stabilization, and long term tensile and flexural modulus properties.

2.1.2. DEFECTS IN CONCRETE CULVERTS (PRECAST & CAST-IN-PLACE; CIRCULAR, BOX & OTHER SHAPES)

2.1.2.1 Introduction

Defects described in this section can be found in both precast and cast in place concrete culverts, regardless of their shape. Precast concrete culverts made from non-reinforced concrete obviously do not develop defects related to steel reinforcement (e.g., corrosion of steel reinforcement, slabbing, etc). The review of defects in this section is based largely on Ballinger and Drake (1995) and Caltrans (2003), however, others sources have been used and referenced.

2.1.2.2 Cracking (Flexure, Shear, Temperature, Shrinkage, Chemical Reactions in Concrete)

Cracks in concrete pipes can be a result of flexure or shear stresses induced by from overloading (structural cracks are active only if the overload condition is continued or if settlement is occurring), thermal expansion and contraction (these cracks tend to widen and narrow with the cycles of the ambient temperature), natural shrinkage of the concrete during curing (these cracks are dormant and are not changing with time), and chemical reactions in concrete (e.g., alkali-silica reactivity typically takes 5 to 12 years to develop cracks).

Longitudinal cracks (Figure 4) are typically found in the crown and invert. Cracks more than 2.5 mm in width may indicate overloading, loss of soil support or poor bedding. Cracking progresses in three stages depending on the bending moment generated in the pipe. Initially, cracks appear at or near the crown, invert and springlines, but the pipe remains supported and held in position by the soil. In the next stage, cracks develop into fractures. Finally, pipe sides move further outwards, and the crown drops once the deformation exceeds 10% (Ballinger and Drake, 1995).

Circumferential (transverse) cracking (Figure 5) occurs typically near the midpoint of the pipe segment, either across the invert of the pipe when the pipe is supported only at ends of each section, or across the crown of the pipe at the midpoint of a pipe segment when there is no adequate depth of suitable bedding over the hard foundation. Cracks in the sides may be caused by settlement or earth pressure (Ballinger and Drake, 1995).

Figure 6 shows cracking of concrete associated with alkali-silica reactivity (ASR): there is one prominent crack while finer microcracks interconnect in random fashion. ASR is a chemical reaction involving alkali hydroxides, usually derived from the alkali present in the cement used, and reactive forms of silica present within aggregate particles. The reaction requires also water to produce the alkali-silica gel, which swells causing cracking of the concrete. The reaction typically takes 5 to 12 years to develop (Swamy, 1998). Drying shrinkage may have contributed to the cracking shown in the photo.

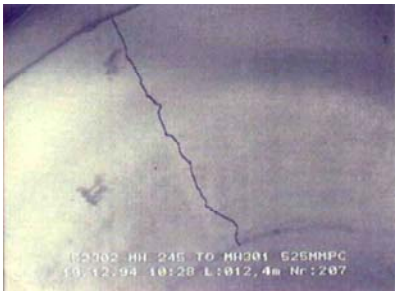


Figure 4. Longitudinal cracking (Caltrans, 2003)

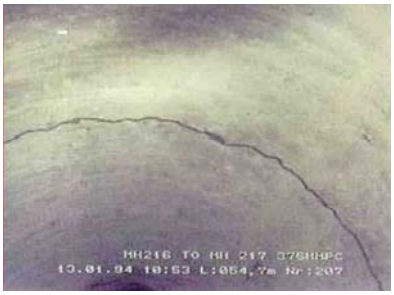


Figure 5. Circumferential cracking (Caltrans, 2003)

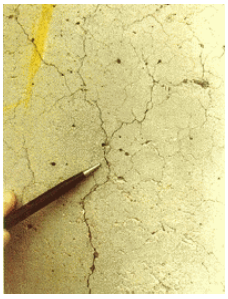


Figure 6. Cracking associated with ASR (AASHTO, 2000)

Internal sulfate attack on concrete may cause secondary ettringite formation (SEF) and lead to concrete cracking, spalling, increased permeability, and strength loss. SEF is a phenomenon by which concrete, normally at ages of a few years or more, is damaged by the gradual formation of ettringite (sulphoaluminate) within the microstructure of the material (Day, 1992). The evidence of ettringite in deteriorated concrete is usually found in larger voids and cracks, but especially in the 'transition zone' (the aggregate/paste-matrix interface) where the porosity is higher than in the bulk matrix and where sulphates are more concentrated. However, most cast-in-place concrete structures with the evidence of SEF appear to be damaged by a combination of factors (alkali-aggregate reactions, corrosion and sulphate attack), while ettringite is believed to be only a contributory cause of destruction. External sulfate attack may also cause SEF instigating substantial destruction if an ample source of external sulphates is provided, e.g., in the soil envelope around the pipe (Day, 1992).

From the point of view of serviceability, the development of cracks with active leaks can be considered the end of maintenance-free performance. From the point of view of structural stability, a cracked section of an unreinforced concrete pipe, with or without active leaks, may be stable as compression arches (unsymmetrical external loading may cause the otherwise stable arch sections to collapse). In effect, the fractured pipe behavior is like a flexible culvert (Law and Moore, 2007). A cracked, reinforced concrete pipe is not a concern unless corrosion of the reinforcing steel occurs at the crack (Zhao et al., 1998).

2.1.2.3 Spalling

Spalling is a fracture of the concrete mass parallel or inclined to its surface. In precast concrete pipe, spalls often occur along the edges of either longitudinal or transverse cracks when the crack is due to overloading or poor support rather than simple tension cracking (Ballinger and Drake, 1995). Spalls show as the breaking off of small pieces of concrete from the concrete surface, and they often appear as circular or oval depressions on surfaces (Figure 7) or as elongated cavities along joints (Figure 8, Figure 9). Spalls can be 1 in. or more deep, and 6 in. to several feet in diameter, although smaller spalls also occur (PCA, 2001). Spall size often depends on culver diameter.

Spalls are caused by pressure or expansion within the concrete, bond failure in two-course construction (new concrete poured over old concrete), impact loads, fire, or weathering (PCA, 2001). In precast concrete culverts, spalls may occur along the edges of cracks that had been caused by overloading or poor support (rather than tension cracking) or may be caused by corrosion of steel reinforcing when water reaches the steel through the cracks (Ballinger and Drake, 1995). If unrepaired, spalls can accelerate concrete deterioration.

Figure 7. Light spalling that can be removed with fingers (Linabond, 2008)

Figure 8. Circumferential spalling (Linabond, 2008)

Figure 9. Spalling at the joint that is open more than 2 in.(Linabond, 2008)

2.1.2.4 Misalignment (Vertical and Horizontal)

In rigid concrete culverts, misalignment occurs at pipe joints when pipe sections cease to be lined up relative to each other and exhibit a disjuncture (offset joints, Figure 10). Vertical misalignment includes sags, heaving and faulting (pipe sections have a difference in elevation i.e., a drop or a step, Figure 11). Horizontal misalignment entails impaired straightness or smooth curvature for culverts constructed with a curved alignment. Misalignment can be caused by improper installation, undermining, uneven settlement of fill, or soil infiltration into the culvert through an opened joint.

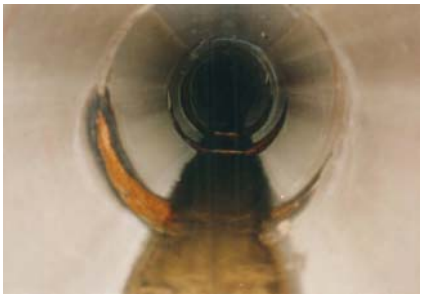


Figure 10. Vertical misalignment. Every joint is leaking (Courtesy of CNA Consulting Engineers)

Figure 11. Section of pipe dropped at the joint so much that a rock hammer fits in the gap (Courtesy of CNA Consulting Engineers)

2.1.2.5 Defective Joints (Cracks and Joint Separation)

Typical joint defects include cracks and joint separation. Cracking and spalling (Figure 9) can develop at the joints between individual segments of concrete pipes. Precast concrete culverts have a natural weakness at the joints. Bell-and-spigot joints can break causing joint offset and infiltration (joint breakage often occurs as a result of damage during construction). Reinforced concrete pipe bells with O-ring gaskets may break if the gaskets are too large or poorly installed.

Separated (open) joint refers to pipe sections that are aligned but separated (displaced longitudinally or the mortar between adjacent concrete sections is lost). This defect is considered severe when the soil outside the pipe is visible (Figure 12).

Defective joints allow infiltration of groundwater into the culvert (Figure 10), or exfiltration into the surrounding soil. In addition, the infiltration of soil into the culvert can occur. The consequences include

undermining, accumulation of debris in the culvert, increased abrasion and the formation of soil voids beneath the driving surface.

Figure 12. Separated joints (Courtesy of CNA Consulting Engineers)

2.1.2.6 End Section Drop-Off

This type of distress, usually due to outlet erosion, is caused by erosion of the material supporting the pipe sections on the outlet end of the culvert barrel. It also may be the result of piping, a process of subsurface erosion where seepage along the culvert barrel removes supporting bedding material.

2.1.2.7 Slabbing (Shear Slabbing or Slab Shear)

Slabbing is a radial failure in concrete pipes that occurs when the curved reinforcing steel straightens and large slabs of concrete “peel” away from the sides of the pipe. The defect is caused by excessive pipe deflection, shear cracks, or in some cases due to inadequate concrete cover.

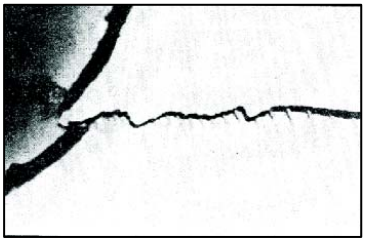


Figure 13. Slabbing (Arnoult, 1986)

2.1.2.8 Undermining and Soil Erosion

Undermining is the loss of soil structural support for the culvert and is manifested as voids in the embedment soil below and around the culvert. This defect can lead to misalignment of concrete culvert segments, as well as cracking and separation of joints. Undermining can be caused by piping, which is subsurface erosion from a flow outside of the culvert that carries away soil around or below the structure (Figure 14), water exfiltration, or infiltration of backfill material under hydrostatic pressure (Figure 15). Medium to large size voids can be formed in the ground as result of the infiltration of rain water into the leaking pipelines, and the washing away of soil in the vicinity (Figure 16). The presence of soil voids can result in ground collapse causing damage to the infrastructure present directly above. For instance, USDA Forest Service (2008) reported that a 24 in. hole had formed over the aged culvert in the

pavement above the South Saint Vrain Creek Creation making safe passage for vehicles impossible and requiring the culvert replacement. Some other examples of sinkholes creation are listed in Jaganathan et al. (2009), while yet other cases can be seen on the web site http://www.sewerhistory.org/grfx/misc/disaster.htm.

Figure 14. Stream flowing underneath the culvert as the soil beneath the culvert has been washed away (Uretek USA, 2008).

Figure 15. Backfill infiltrating through a joint opening (Mitchell et al., 2005)

Figure 16. A still image extracted from a video clip about soil voids creation around leaking pipes (Insituform, 2009)

2.1.2.9 Corrosion and Abrasion

Metallic corrosion in reinforced concrete is an electrochemical reaction of reinforcing steel embedded in moist concrete: free water present in the pores of concrete contains various dissolved ions and serves as the electrolyte, iron dissolves and rust forms at the cathode (Figure 17). Corrosion requires the following conditions to take place (Carino, 1999): (1) loss of passivation (with the initial pH normally about 12.5 in the pores of concrete, a passive oxide coating is formed on the steel reinforcement because of alkaline conditions, which is gradually lowered due to carbonation or presence of chloride ions above the critical concentration), (2) presence of moisture (water penetrates the concrete through pores, cracks such as shown in Figure 18 or spalls), and (3) presence of oxygen.

Chloride ions causing the loss of passivation can be found in severe environments, e.g., where sea water or deicing salts are present (NRMCA, 1998). The use of de-icing salts on roadways is probably one of the primary causes of premature failure of reinforced concrete culverts.

Another cause of steel corrosion is carbonation of concrete (a chemical reaction between carbon dioxide in the air and hydroxides, principally calcium hydroxide but also calcium silicate and aluminates, to form carbonates). This reaction reduces the pH of the pore solution to as low as 8.5, at which level the passive film on the steel is not stable. The rate of carbonation is dependent on the relative humidity of the concrete (the highest rates occur when the relative humidity is between 50% and 75%). Also, if the concrete is cracked, the carbon dioxide in the air can penetrate easier into the concrete matrix. Carbonation of concrete also lowers the amount of chloride ions needed to promote corrosion (PCA, 2010).

PCA (2002) provided a clear and concise overview of each of the types and causes of concrete deterioration, including corrosion of embedded metals.

Further discussion of aggressive chemicals attacking the steel reinforcement is given in 2.1.3.2.

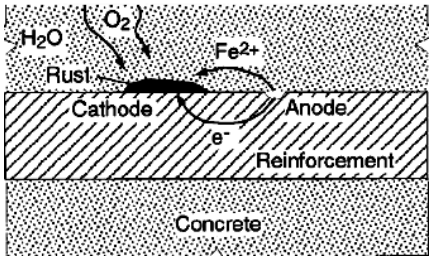


Figure 17. Corrosion of steel in concrete (Carino, 1999)

Figure 18. Stains from a longitudinal crack indicate corrosion of steel reinforcement (Courtesy of CNA Consulting Engineers)

Figure 19. Abrasive, angular quartz in a sand bedload (Caltrans, 2003)

Non-metallic corrosion also takes place in concrete culverts. Gérard et al. (1999) reviewed the fundamentals that govern the durability of cement-based materials in contact with water. Deterioration of concrete structures subjected to aggressive water is characterized by the leaching of calcium from hydrates, which is essentially a diffusion-controlled phenomenon. The main chemical component of the binder is calcium, present both in solids phases (in the form of hydrated calcium silicates C-S-H and portlandite crystals $Ca(OH)_2$) and in the form of calcium ions in the pore solution. At high concentration of calcium ions in the pore solution, C-S-H and $Ca(OH)_2$ remain insoluble. As this concentration drops below certain critical level, $Ca(OH)_2$ dissolves first. Next, C-S-H undergoes partial decalcification. Beyond a certain critical level, even the C-S-H becomes totally decalcified and the remaining product is a silica gel without any binding properties. The main consequences of calcium leaching on the cement paste microstructure are an increase in porosity, a loss of elastic properties and a loss of strength.

Janakiram and Vipulanandan (1998) carried out a laboratory testing to examine the leaching of calcium in acidic and sulfate environment. Gérard et al. (2006) developed a numerical model to predict concrete deterioration and the residual service life of structures while treating calcium leaching as a nonlinear phenomenon influenced by a wide range of parameters.

Abrasion is the wearing away of concrete surface by bed load, i.e. sediment and debris being transported by the flow, by rolling, sliding, or skipping along the bed or very close to it. Multiple factors affect the abrasion potential of a site and associated service life of a culvert: size, shape, hardness and volume of bed load, as well as volume, velocity, duration and frequency of flow in the culvert (Caltrans, 2003). An example of an estimator of abrasion potential in a culvert is shown in Table 3, however it is based only on two factors (bed load size and flow velocity) whereas other factors should also be taken into consideration. For instance, heavy loads of hard, angular sand (Figure 19) can be more abrasive than small volumes of relatively soft, large rocks. Two sites with similar site characteristics but different hydrologic characteristics (i.e.,, volume, duration and frequency of flow in the culvert) will have different abrasion levels.

Corrosion and abrasion can cause further spalling and cracking, as well as scaling and delamination (see section 2.1.2.10).

Table 3. Preliminary estimator of abrasion potential (after Caltrans, 2003)

Abrasion level	Bed load size	Flow velocity
Non abrasive	Little or no bed load	Less than 1 m/s
Low abrasive	Minor bed loads of sand, silts, and clays	Less than 1.5 m/s
Moderate abrasive A	Moderate bed loads of sands, gravels, and small cobbles with max stone sizes up to about 150 mm.	From 1.5 m/s to 3 m/s
Moderate abrasive B	Moderate bed loads of sands, gravels, and small cobbles with max stone sizes up to about 150 mm. For larger stone sizes within this velocity range, see 'Severe Abrasive'	From 3 m/s to 4.5 m/s
Severe abrasive	Heavy bed loads of sands, gravel and rocks, with stone sizes 150 mm or larger	Greater than 3 m/s

2.1.2.10 Concrete Surface Defects (Scaling, Delamination, Popouts, Efflorescence)

Scaling is local flaking or peeling of concrete surface as a result of chemical breakdown of the hardened cement paste (chemicals such as ammonium sulfate or ammonium nitrate, which are components of most fertilizers, can induce severe chemical attack on the concrete surface and cause scaling), or exposure to freezing and thawing. Light scaling does not expose the coarse aggregate. Moderate scaling involves loss between $^{1}/_{8}$ and $^{3}/_{8}$ inch of the surface mortar starting to expose the aggregate. In severe scaling, the aggregate is clearly exposed. Long-term scaling leads to the gradual and continuing loss of aggregate over an area (NRMCA, 1998 "CIP 02").

Delamination is the separation of a surface layer from the main body of concrete. It is often caused by bleed water and bleed air that remains trapped below the prematurely densified mortar surface (usually top $^{1}/_{8}$ to $^{3}/_{8}$ inch of the surface mortar gets delaminated). Delamination may also be caused by disruptive stresses from chloride-induced corrosion of steel reinforcement or of poorly bonded areas in two-course construction. These delaminations are deeper than those caused by trapped air or bleed water (NRMCA, 1998 "CIP 20").

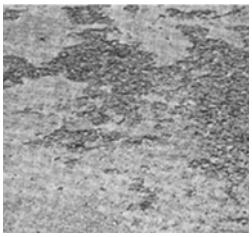


Figure 20. Scaling (NRMCA, 1998)

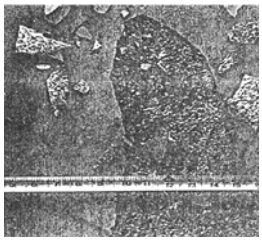


Figure 21. Delamination (NRMCA, 1998)

Figure 22. Honeycombs (CCAA, 2001)

Figure 23. Pop-outs (NRMCA, 1998)

Figure 24. Crack with efflorescence (RJ Burnside & Assoc, 2005)

Honeycombing refers to voids in concrete caused by the mortar not filling the spaces between the coarse aggregate particles. Depending on the depth and extent, honeycombing may reduce both the durability performance and the structural strength of the concrete. Honeycombing is caused either by compaction not having been adequate to cause the mortar to fill the voids between the coarse aggregate, too low of a cement content, or by holes and gaps in the formwork allowing some of the mortar to drain out of the concrete (CCAA, 2001)

Popouts are conical fragments that break out of the concrete surface leaving small holes, which typically occur because the concrete has been overworked, allowing aggregates to drift upward toward the surface (NRMCA, 1998 "CIP 40"). Another cause is corrosion of the steel reinforcement resulting in volume expansion of the steel (approximately 2%) and internal pressures.

Efflorescence is a white crystalline or powdery deposit on the surface of the concrete. The deposit is a combination of calcium carbonate leached out of the cement paste and other recrystalized carbonate and chloride compounds. It starts as a salt being dissolved by water; the salt solution is then transported by

gravity or by capillary action to a surface exposed to air where the solution evaporates and leaves behind the crystalline deposit (Goldberg, 1997). Thus, efflorescence requires three simultaneous conditions to occur: presence of soluble salts (e.g.,, calcium chloride contamination in the concrete from deicing salts, acid rain, or contaminated water used in the concrete mix), presence of water (for extended period), and transporting force (gravity, capillary action, hydrostatic pressure, evaporation).

Efflorescence starts as a cosmetic annoyance, however if the fundamental cause (water infiltration into concrete) is left uncorrected, continued efflorescence can become a functional defect. The primary danger is potential bond failure resulting from continued depletion of calcium and subsequent loss of strength of cementitious adhesives and underlying cementitious components. The crystallization of soluble salts can exert more pressure than the volume expansion forces caused by ice formation. This mechanism may result in spalling or bond failure (Goldberg, 1997).

2.1.2.11 Summary of Defects in Concrete Culvert Pipes

Defect types in concrete culvert pipes and their causes are summarized in Table 4.

Table 4. Summary of defects in concrete culverts

Defect Type	Description	Cause
Cracking	Longitudinal and circumferential cracks typically	Flexure, shear (also temperature, shrinkage, alkali-silica reactivity, secondary ettringite formation)
Spalling	Circular or oval depression on concrete surface or elongated cavities along the joints	Pressure or expansion in concrete, bond failure in 2 course construction, impact loads, etc
Misalignment	Sags, heaving and faulting; impaired straightness	Settlement, undermining, improper installation
Defective joints	Cracks and joint separation	Settlement, undermining, improper installation
End section drop-off	Dropping of the end section	Soil erosion at the outlet end, piping
Slabbing	Straightening of curved reinforcing steel and “peeling” away of large concrete slabs from the pipe sides	Excessive pipe deflection, shear cracking
Undermining	Voids in the soil bedding below or around the culvert	Piping, water exfiltration, backfill infiltration into culvert
Metallic corrosion	Deterioration of steel reinforcement	High chloride content in the environment (use of de-icing salts on roadways), carbonation of concrete
Non-metallic corrosion	Deterioration of hardened concrete paste	Aggressive chemicals attacking hardened concrete paste present in the flow or groundwater; calcium leaching from the cement binder
Abrasion	Flowing water erodes the pipe inner surface	Flowing water contains abrasive sediment, debris
Scaling	Local flaking or peeling of concrete surface, loss of aggregate over an area	Corrosion of hardened cement paste, exposure to freezing and thawing
Delamination	Subsurface separation of concrete into layers	Poor installation (prematurely densified mortar surface), corrosion of steel reinforcement
Honeycombing	Voids between the coarse aggregate particles	Poor installation (improper compaction, poor formwork)
Pop-outs	Small holes in the concrete surface created when a conical fragment breaks out	Overworking of concrete, corrosion of steel reinforcement
Efflorescence	White crystalline deposit on the surface potentially causing bond failure in concrete	Soluble salts in concrete (calcium chloride contamination) and presence of water for prolonged time

2.1.3. DEFECTS IN METAL CULVERTS

2.1.3.1 Introduction

Defects described in this section can be found in corrugated metal pipes (corrugated aluminum and corrugated steel pipes) and corrugated metal structural plate culverts. The review is mostly based on the *Culvert Inspection Manual* of Arnoult (1986), however, others sources have also been used and referenced.

2.1.3.2 Corrosion and Abrasion

Corrosion. The majority of metal pipe failures can be attributed to corrosion, which can attack the inside or outside of culvert barrels. The aggressive chemicals can be in the flowing water (the damage is more serious in culverts with continuous flows or standing water than with intermittent flows) or in the groundwater, and they can originate in the soil, be introduced through contaminants in the backfill soil, or be transported by surface or subsurface flows. Soil and water conditions that are particularly aggressive or hostile to culverts are those with pH values of less than 5.0 (strongly acid) or greater than 8.5 (strongly alkaline). Water acidity can be mineral or organic. Mineral acidity comes from sulfurous wells and springs, and drainage from coal or other mines; the water contains dissolved sulfur and iron sulfide that may form sulfurous and sulfuric acids. Mineral acidity with a pH as strong as 2.3 has been encountered. Organic acidity, which may be found in swampy land and barnyards, may have a pH as low as 4.0. Alkalinity in water is caused by strong minerals and limed and fertilized fields.

Abrasion is the wearing away of the inside surface of corrugated metal pipes, primarily at the pipe invert, by sediment and debris being transported by the flow. Overall, factors affecting the abrasion potential in concrete culverts (see 2.1.2.4) are valid to all pipe materials. However, the wear of material in corrugated metal pipes is greater than in smooth surface pipes because there is additional detrimental effect of abrasive material striking the upstream face of corrugations (protective coating materials applied in corrugated metal pipes typically do not fill the corrugations).

Minor corrosion and abrasion can cause delamination of protective coating (Figure 25), pitting and perforation holes (Figure 26, Figure 27). Major corrosion can cause structural failure and collapse.

Figure 25. Delaminated protective coating (Mitchell et al., 2005, detail)

Figure 26. Holes in the corrugated steel pipe from corrosion (Caltrans, 2003, detail)

Figure 27. Worn invert due to abrasion (Caltrans, 2003, detail)

Apart from the obvious failure from a collapse, a culvert pipe that has a corroded or abraded invert, or a pipe that is severely pitted and perforated, may still be capable of supporting its backfill and cover but it constitutes a high risk and requires a prompt repair or replacement (NCHRP, 1978).

2.1.3.3 Shape Distortion (Ovality, Flattening, Peaking, Loss of Symmetry) and Buckling

Shape changes in flexible metal culverts indicate excessive load and/or inadequate backfill or embankment support for the culvert. Round or elongated pipes can settle and exhibit an increase in the horizontal span (ovality). A slightly increased horizontal span may be created during the construction and is not detrimental, but a significant increase may be associated with low stiffness backfill soil, and this low stiffness backfill can also compromise the culvert's resistance to global buckling, so collapse occurs (Figure 28). While round pipes tend to deflect more uniformly, arches have a tendency to peak during backfilling or to rack (deflect) to one side causing loss of symmetry (Figure 29) and potentially collapse.

Metal culverts can also exhibit side flattening (at shallow depth, soil pressure on the sides is greater than on the top of culvert), flattening of the top arc, and flattening of the bottom arc. While minor flattening is not serious, points where the radius of curvature changes sign should be monitored to ensure the structure does not continue to deform until collapse. Pipe arches, which transmit load to the foundation principally at the points of reaction near its base ("corners"), may have inadequate support at these corners causing the reaction points to sink and spread while the invert stays in place, so that it appears as if the bottom was pushed up (Figure 30).

Figure 28. Collapsed corrugated metal culvert (Insituform Technologies, 2006)

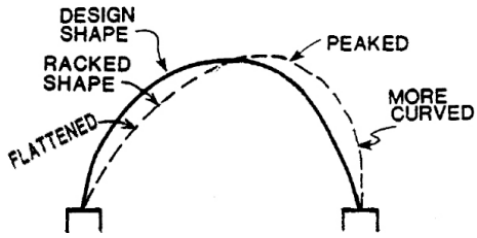


Figure 29. Racked and peaked arch (Arnoult, 1986)

Figure 30. Settlement and invert distortion of pipe arches (Arnoult, 1986)

2.1.3.4 Undermining (Erosion Voids)

A normal outcome of metal culvert pipe corrosion that has produced perforation is undermining, i.e. the formation of erosion voids in the embedment soil below and beside the structure (Figure 31, Figure 32), typically adjacent to the location of severe corrosion, due to ingress of water through the corroded zones and infiltration of backfill material under hydrostatic pressure (Figure 33). Undermining represents a loss of soil support for the culvert. Without an adequate soil envelope around the pipe, metal culverts have relatively little bending stiffness and are likely to deflect and exhibit shape distortion or sags, and joint opening. Erosion voids can spread and eventually reach the ground surface having an enormous impact on the stability of buried metal culverts, in addition to potential loss of life if a void forms under a pavement.

El Taher and Moore (2009) investigated the impact of backfill soil erosion on corrugated metal culvert stability against buckling. The study was limited to shallow-buried culverts under two boundary conditions (symmetrical and unsymmetrical erosion). The study showed that 1) the stability against elastic buckling is jeopardized more severely when erosion patterns are symmetric though one-sided and symmetric erosion compromises structural stability, 2) increases in the erosion void volume (or culvert perimeter without soil support) and the extent of invert corrosion lead to greater decreases in elastic buckling strength, and 3) deflections in the eroded structure increase only moderately (less than 2.5% in the study), so culvert stability cannot be reviewed simply by monitoring culvert shape.

Figure 31. A large void space under culvert (Mitchell et al., 2005)

Figure 32. Bedding washed out under the culvert from the outlet end to about 13 ft away (Mitchell et al., 2005)

Figure 33. Bad seams that allow leakage and undermining of backfill (Wyant, 2002)

2.1.3.5 Defective Joints

Several types of defective joints occur in corrugated metal culverts:

- Separated (open) joints: Pipe sections are aligned but separated (displaced longitudinally)
- Misaligned (offset) joints: Pipe sections are not lined up relative to each other (vertically and horizontally) and exhibit a disjuncture. Misalignment can be caused by improper installation, undermining, uneven settlement of fill, or soil infiltration into the culvert through an opened joint).
- Faulting joints: Adjacent pipe sections have a difference in elevation (a drop or a step).
- Partially open joints: Pipe sections remain connected and aligned, but a partial joint opening is created due to the shape distortion of one pipe section (typically a drop in the pipe crown is visible) or a relative rotation of one segment with respect to the joint plane.

2.1.3.6 Sags/Heaving

Corrugated metal culverts can also have sags or heaving due to soil settlement and poor backfill support. Minor sags or heaving are generally not a significant problem in corrugated metal structures unless it causes shape or joint problems.

2.1.3.7 End Section Defects (Buckling, Undermining)

Corrugated metal culverts with mitered ends (cut to match the embankment slope) and ends of skewed culverts (the end cut parallel to the centerline of the culvert) have reduced strength in these areas and are more susceptible to buckling, Moore et al. (1995a, 1995b). Culvert ends beyond the embankment are likely exposed to greater erosion and piping and therefore more susceptible to undermining (ConnDOT, 2000).

2.1.3.8 Seam Defects (Longitudinal and Circumferential)

Longitudinal seams (found in structural plate culverts) can have defects in the form of open seams, cracking at bolt holes, plate distortion around the bolts, bolt tipping, cocked seams (Figure 34), cusped seams (Figure 35), and significant metal loss in the fasteners due to corrosion. The circumferential seams are rarely damaged but if they are, they are either open (e.g., when a culvert on a steep slope is pulled apart longitudinally) or vertically misaligned due to a foundation failure.

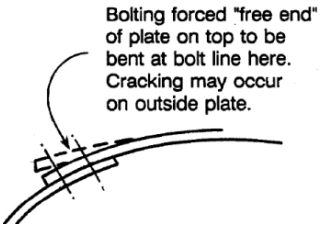


Figure 34. Cocked seam (Arnoult, 1986)

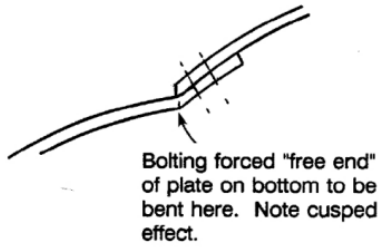


Figure 35. Cusped seam (Arnoult, 1986)

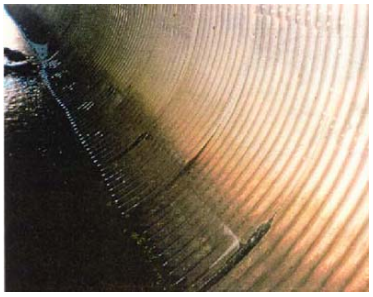


Figure 36. Bad longitudinal seam (Wyant, 2002, detail)

2.1.3.9 Cracks, Dents and Other Localized Damage

Defects in metal pipe wall such as creases, bulges, dents, cracks, pinholes (Figure 37), large holes (Figure 38), or tears can impair the structural integrity of the barrel in ring compression or permit infiltration of backfill. Dents in corrugated steel culverts may be accompanied by cracking or debonding of the protective coating (Figure 25).

Figure 37. Seepage water flowing in through pinhole (Mitchell et al., 2005, detail)

Figure 38. Elongated holes with gray backfill soil visible (Mitchell et al., 2005, detail)

2.1.3.10 Footing Defects

Metal culverts built on concrete footings (e.g., corrugated metal structural plate arches, metal long-span culverts) can develop defects such as wrinkling, cracking, spalling, compression or stretching of the corrugations in the culvert barrel due to differential footing settlement (one footing section settles more than the rest of the footing) or rotation of footing due to undermining (Figure 40). Flexible corrugated metal culverts can tolerate some differential settlement but get damaged from extensive ones. The severity of damage increases with the degree of differential footing settlement, decrease in the length over which it is spread, and decrease in the arch height (Figure 39).

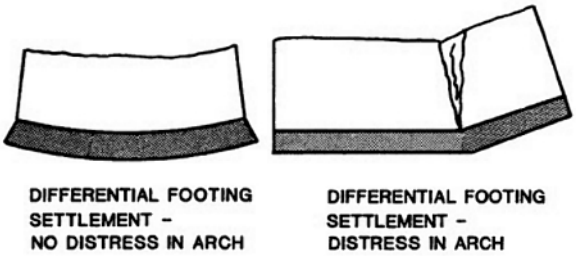


Figure 39. Differential footing settlement (Arnoult, 1986)

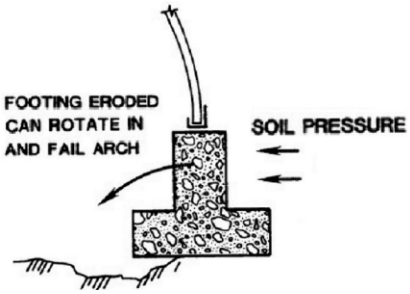


Figure 40. Rotation of footing due to undermining (Arnoult, 1986)

2.1.3.11 Summary of Defects in Corrugated Metal Culvert Pipes

Defect types in corrugated metal culvert pipes and their causes are summarized in Table 5.

Table 5. Summary of defects in corrugated metal culverts

Defect Type	Description	Cause
Corrosion	Delamination of protective coating, pitting, perforations	Water conditions with pH less than 5.0 or greater than 8.5
Abrasion	Flowing water erodes the pipe inner surface, especially invert	Flowing water contains abrasive sediment, debris
Shape distortion	Settlement, flattening, peaking, loss of symmetry	Excessive load and/or inadequate backfill support
Undermining (erosion voids)	Voids in the soil below or around the culvert	Backfill infiltration into the culvert through corroded or missing sections of the pipe under groundwater pressure
Defective joints	Separated or misaligned joints resulting in exfiltration and infiltration	Settlement, undermining or improper installation
Sags or heaving	Vertical misalignment between joints	Soil settlement, poor backfill support
End section defects	Buckling, undermining	Piping, water exfiltration, backfill infiltration into culvert
Seam defects	Defects in longitudinal seams and circumferential seams	Corrosion, foundation failure, pulling apart of culvert sections on vertical slopes
Localized damage	Cracks, dents	Corrosion, abrasion, problems in backfill support
Footing defects	Differential footing settlement, rotation of footing	Problems in backfill support , undermining

2.1.4. DEFECTS IN Thermoplastic Culverts

2.1.4.1 Introduction

Defects described in this section can be found in thermoplastic culverts, which are usually made of high-density polyethylene (HDPE) pipe with corrugated or other profiles such as square, tubular (a.k.a. honeycomb), or corrugated or ribbed PVC. In profiled wall thermoplastic pipes (see section 1.2), the type and location of some of the defects depend on the geometry of the wall profile.

2.1.4.2 Cracking

Cracks in thermoplastic culverts can be circumferential (Figure 41, Figure 42), longitudinal, or mixed. In addition, elliptical cracks can develop around buckles (discussed later in this section).

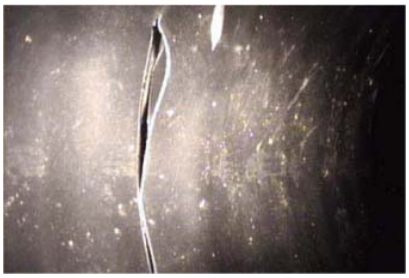


Figure 41. Ring crack in an HDPE pipe (Abolmaali and Motahari, 2008)

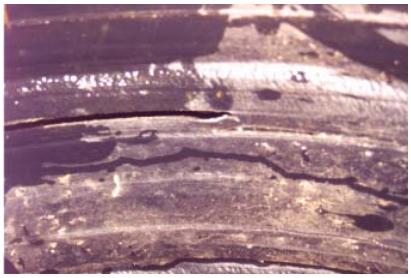


Figure 42. Ring crack in a 30 in. HDPE pipe, from 80° to 110°, at footage 10 ft (Nelson and Krauss, 2002)

Hsuan and McGrath (1999) investigated field retrieved samples from cracked HDPE culvert pipes throughout the US (obtained from 29 field sites that had been identified from a questionnaire sent out to 50 state DOTs, federal and local agencies and engineering consultants). The dominant type of cracking was circumferential cracks, although longitudinal cracking was also observed in some sites.

Cracking in thermoplastic pipes is identified by the separation of two surfaces. Two types of cracking failures can be distinguished: ductile and brittle.

Ductile failure mode is primarily driven by the yield stress of the polymer or pipe (Krishnaswam, 2005). The material undergoes extensive plastic deformation before fracture: a uniform deformation of fibers occurs resulting in high elongations at the crack tip (Lustiger and Corneliussen, 1987). The fracture surface consists of many micro-voids and dimples, and the pipe wall thickness along the fracture surface is typically reduced. However, this type of failure is not of primary concern in thermoplastic culverts as they appear to fail predominantly in brittle manner: all analyzed cracks in the study by Hsuan and McGrath (1999) occurred in the brittle-like manner.

In brittle cracking, there is relatively little deformation and material flow in the crack area and the fracture surface is visibly smooth. Two fundamentally different failure modes can be associated with brittle cracks (termed “brittle cracks” due to lack of any visible ductility in the crack area): (1) rapid crack propagation, with crack-growth rate close to speed of sound, and caused by impact-type loading, and (2) stress cracking, developed under long-term low-level loading conditions, with crack propagating over relatively long time period (several hours to several years). In polyethylene, impact failure tends to occur at lower temperatures and higher loads, and stress cracking at higher temperatures and lower loads (Lustiger and Corneliussen, 1987).

Although both rapid crack propagation and stress cracking yield visibly smooth fracture surface, the fracture surface is very different when inspected microscopically using a scanning electron microscope (SEM). Impact loading creates the fracture surface with a flaky, scaly appearance (Figure 43), whereas stress cracking creates a fibrous morphology (Figure 44). In the study by Hsuan and McGrath (1999), the majority of analyzed crack surfaces exhibited a fibrous structure, indicating slow crack growth mechanism. Only in few pipes the fracture surface showed a flake structure indicating rapid crack propagation mechanism.

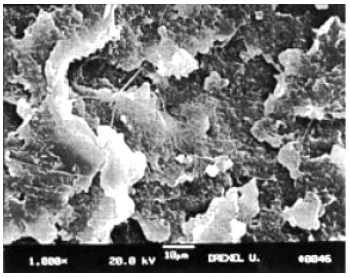


Figure 43. Flake morphology characteristic for impact-type fracture (Hsuan and McGrath, 1999)

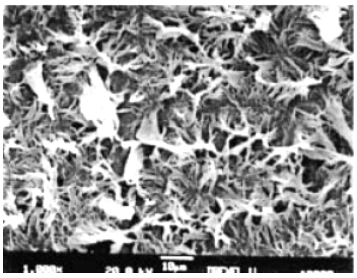


Figure 44. Fibrous morphology characteristic for slow crack growth (Hsuan and McGrath, 1999)

Slow crack growth (SCG) failure mechanism occurs due to the fact that thermoplastics, when subjected over a long time period to tensile stresses substantially lower than those necessary to cause a short-term rupture, can develop crazes and small cracks that grow slowly until eventually rupture occurs. Crazes are “very fine cracks” developing in the direction normal to tensile stress that have the surfaces still bounded together by molecular fibrils, approx 10 nm in diameter, which continue to support the load (i.e., crazes represent the redistribution of local stresses throughout the thermoplastic matrix). The formation of these crazes and cracks is not caused by any chemical degradation of polymer and is only the result of mechanical and/or thermal forces (Mruk, 1999). In general, the rate of SCG can be accelerated by different factors, e.g., stress intensity, cycling of the stress (fatigue), elevated

temperature, and exposure to certain environments (referred to as environmental stress cracking).

This type of brittle cracking in HDPE pipes generally results from a combination of high tensile stress (due to applied loads, residual stresses, or thermal effects), and low quality resin with poor crack resistance. It may be associated with simple two dimensional behavior of the pipe (where circumferential tensions develop on the outside surface of the pipe at the springlines, or on the inside surface at the crown or invert). More commonly, the tensile stresses result from three dimensional behavior caused by complexities of the profile (e.g., Moore and Hu, 1995).

An example of culvert cracking due to SCG failure mechanism is a circumferential crack found in Type S-helical HDPE pipe (Hsuan and McGrath, 1999, Site F). The pipe (42 in. ID) experienced deflection (11% vertical diameter decrease and 10% horizontal diameter increase), as well as buckling and cracking (one longitudinal and several circumferential cracks), as shown in Figure 46, in an area approx 30 ft from the inlet. Circumferential cracks (between 18 in. and 24 in. long) developed at pipe crown, at the junction between liner and corrugation (Figure 45). The examination of fracture surface showed constant thickness along the fracture surface (i.e., no sign of ductility) and further revealed that the crack had initiated from the outer surface (Figure 47). A close view on the fracture surface revealed fibrous morphology that is associated with a SCG. Such cracking typically occurs very slowly, under static loading. The tensile stress that initiated the crack was likely due to the deflection of the pipe. As the crack was located near the pipe outlet and soil erosion was observed under the pipe, it was concluded that longitudinal bending was the main cause of cracking.

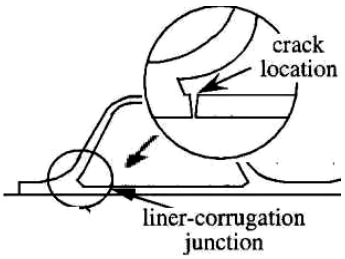


Figure 45. A crack developing at the liner/corrugation junction (redrawn from Hsuan and McGrath, 1999)

Figure 46. Buckling and cracking of the liner in type S-helical HDPE pipe (Hsuan and McGrath, 1999)

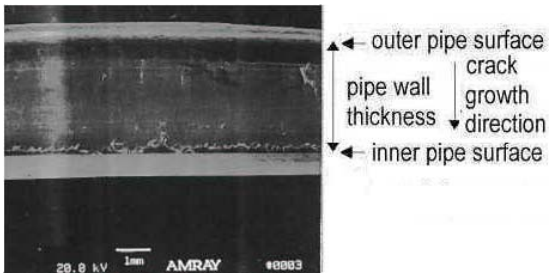


Figure 47. Fracture surface of crack (Hsuan and McGrath, 1999)

The study by Hsuan and McGrath (1999) also showed that pipe geometry of the HDPE corrugated pipe is one of the factors that influence the SCG mechanism in these pipes. In Type C pipes, full circumferential cracking results in separation of the pipe and leads to loss of bedding soil. In Type S pipes, circumferential cracking typically occurs at the junction between the liner and the corrugation. The cracks typically initiate from the outer surface growing through the liner thickness. This type of cracking may not lead to collapse of the pipe, but can adversely affect its hydraulic performance. In Type S- honeycomb pipes (Type D in some AASHTO specifications), both longitudinal and circumferential cracks develop in the inner liner. Longitudinal cracks propagate through the outer corrugation allowing soil to infiltrate into the pipe. This type of cracking can be critical for the long-term soil/pipe interaction.

Another type of cracks that can develop as result of SCG failure mechanism is associated with buckling: elliptical cracks around the buckles. An example of this cracking was found in Type S-honeycomb HDPE pipe (Hsuan and McGrath, 1999, Site E). The pipe (42 in. ID) experienced deflection and localized buckling through much of its length. (The buckling commonly occurred with a short wavelength sinusoidal pattern and within the liner, around the circumference of pipe - see section 2.1.4.3 for discussion on buckling). The majority of cracks occurred around the localized buckling regions (Figure 48). The cracks were initiated from the inner surface and grew through the thickness of the liner. In addition, circumferential splitting cracks occurred at the corrugation/liner intersection (Figure 49).

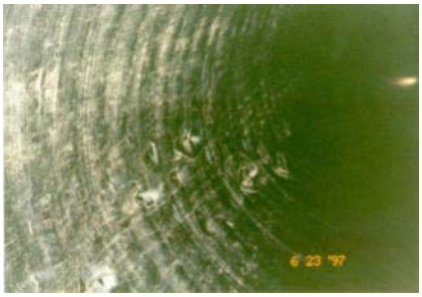


Figure 48. Buckling and elliptical cracking along springline in type S-honeycomb HDPE pipe (Hsuan and McGrath, 1999)

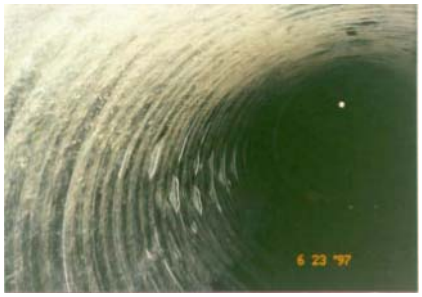


Figure 49. Circumferential cracks in form of splitting cracks along the liner and corrugation (Hsuan and McGrath, 1999)

Rapid crack propagation (RCP) failure mechanism. This failure mechanism is typically associated with a dynamic force and cold temperature. An example of culvert cracking due to RCP failure mechanism is a long circumferential crack found in Type C-helical HDPE pipe, shown in Figure 50 (Hsuan and McGrath, 1999, Site B). The crack occurred at the joint between web and valley. Examination of the fracture surface shows constant thickness along the fracture surface (i.e., no sign of ductility) and further reveals that the crack initiated from the inner surface (Figure 51). A close view on the fracture surface (Figure 52) reveals flake morphology that is associated with a RCP. Such cracking typically occurs very fast, under dynamic loading. The crack in this pipe occurred in the area near the outlet of the pipe, where a significant amount of longitudinal bending caused by soil erosion under the pipe was observed. Thus, longitudinal bending was the main cause of cracking.

Figure 50. Type C-helical HDPE pipe with a crack due to RCP (Hsuan and McGrath, 1999, site B)

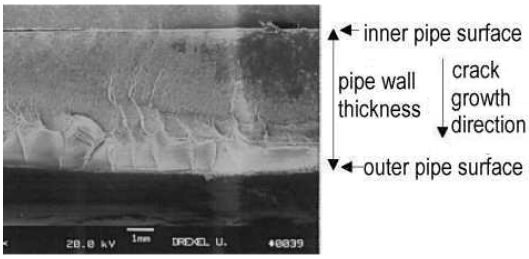


Figure 51. Fracture surface of crack due to RCP (Hsuan and McGrath, 1999)

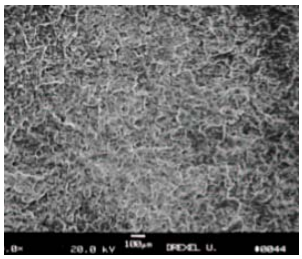


Figure 52. Flake morphology (Hsuan and McGrath, 1999)

Stress crack resistance (SCR). For assessing the stress crack resistance (SCR) of HDPE resins, the environmental stress crack resistance (ESCR) test, ASTM D 1693, included in AASHTO M 294 can be used. However, the ESCR test has some limitations making it unsuitable for evaluating SCR of HDPE pipes (e.g., the failure time of individual test specimens can't be recorded, the large standard deviation value, etc).

Hsuan and McGrath (1999) validated an alternative test, the single-point notched constant tensile load (SP-NCTL) test, ASTM D 5397-Appendix, as a measure of slow crack growth (SCG) resistance under sustained loading of HDPE non-pressure drainage pipes. The results of SP-NCTL test give the failure time of specimens at 15% yield stress. Materials with longer failure times have a greater SCR. Data from field retrieved pipe samples indicated that pipes made from high SCR resins did not exhibit cracking even under large deflection. The study proposed minimum failure times of the SP-NCTL test based on type of pipe profile: 1) 400 hours for type C and type S-helical pipes, and 2) 24 hours for type C and type S-annular pipes and type-S honeycomb pipes. For all types of pipe profiles, setting lower minimum test times for individual test accounts for statistical variability.

Effect of residual stresses on SCR. Residual stresses in HDPE pipes have a profound effect on SCR. Residual stresses are internal stresses that are generated during the manufacturing process: thermoplastic

pipes are extruded as relatively soft and hot tubes, and the subsequent cooling process creates the thermal gradient in the pipe wall, which in turn generates residual stresses in circumferential and longitudinal direction. If the cooling is applied from one side of pipe wall (e.g., from the outside), a compression hoop stress is produced at the outer side of the pipe wall, and a tensile hoop stress in the inner side of the pipe wall (Figure 53). If both sides of pipe wall are cooled simultaneously, tensile stresses are left in the central core section. Several studies focused on residual stresses in uniformly solid smooth wall pipes, e.g., Clutton and Williams (2004) measured the residual stresses in both circumferential and longitudinal direction. In corrugated pipes, however, complex geometry of pipe walls creates complicated residual stresses.

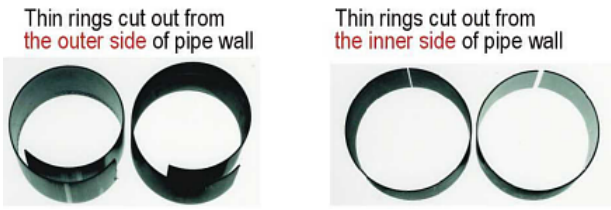


Figure 53. Thin rings (the thickness approx 10% of the pipe wall thickness) cut from HDPE pipe that was cooled from outside during the manufacturing process (Redrawn from Farshad, 2006)

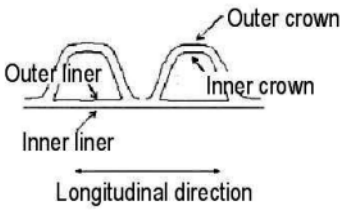


Figure 54. Surfaces in the pipe profile tested in the SCR study (Redrawn from Hsuan and Zhang, 2005)

Hsuan and Zhang (2005) investigated the stress crack resistance (SCR) in profiled HDPE pipes. The study used newly manufactured pipes, thus focusing on the effect of residual stresses on the SCR. The notched constant ligament stress (NCLS) test (ASTM F 2136) was used to evaluate SCR of specimens, which were taken in longitudinal and circumferential direction from different locations of the pipes, as shown in Figure 54 (only circumferential specimens could be taken from the crown because of the limited width and curvature of the crown). In longitudinal direction, the outer surface of the liner had much worse SCR than the inner surface of the liner (i.e., 30.4 hrs versus 88.5 hrs). The tensile residual stress in the outer surface of the pipe liner helped shorten the failure time, whereas the compressive residual stress in the inner surface of the pipe liner resisted the cracking process. The opposite was found in circumferential direction. Circumferential specimens were curved, having compressive stresses in the outer surfaces and tensile stresses in the inner surfaces of both the liner and the crown. Therefore, the outer surfaces of the liner and the crown have better SCR than the corresponding inner surfaces (i.e., 37.0 and 37.8 hours vs. 26.8 and 25.1, respectively).

2.1.4.3 Deflection and Buckling

A primary concern regarding the performance of thermoplastic pipes, especially those used in large diameter culverts, is wall stability. Most defects can be contributed to poor design or installation practices, e.g., inadequate compaction of low quality backfill material (Blackwell and Yin, 2002).

Time dependent deflection of buried plastic pipes. Properly designed and installed buried thermoplastic pipes deflect at the time of installation (initial deflection) and continue to deflect over time until an equilibrium state is reached (the long-term deflection). Provided the envelope of backfill soil surrounding the pipe remains stable (there is no loss of soil due to erosion, for example), pipe deflection remains constant since the soil provides most of the stiffness resisting the soil loads. Changes in secant modulus of the thermoplastic over time then have little net impact on the total stiffness of the soil-pipe system. However, internal stresses in the pipe wall will decrease significantly over the service life of the pipe as a result of stress relaxation.

Three kinds of deflection can be observed in the plane of the pipe normal to the pipe axis. First, the fact that vertical stresses in the ground generally exceed the horizontal earth pressures, all flexible pipes adopt an oval shape (Figure 55) as explained by Spangler (1941). Next, profiled thermoplastic pipes

have low hoop stiffness relative to the soil surrounding them, so circumferential shortening is common (particularly in HDPE pipes), Selig et al. (1994), (Moore, (2001). Third, local variations in soil support around the pipe circumference lead to non-oval deformation patterns like those identified by Rogers (1988) and more recently quantified by Brachman et al. (2008). This means that local bending strains in the haunches of the pipe may be enhanced above those otherwise expected for given levels of diameter change. The procedure of Molin (1981) has recently been suggested for incorporation in AASHTO pipe design, Brachman et al. (2008) and McGrath et al. (2008), where the empirical strain factor D_f is employed. This approach is already used in design of fiberglass pipes, i.e., AWWA M45 (AWWA, 2008).

Longitudinal (beam) deflection is of minor concern when the bedding is good.

Use of low stiffness (improper) bedding can reduce flexible pipe resistance to global buckling, and buckling may then develop. This might be at the pipe crown (Figure 56) particularly when the pipe is at shallow cover. In extreme cases, global buckling can ultimately lead to pipe collapse (Figure 57).

Figure 55. Ovality in a 42 in. HDPE pipe (Nelson and Krauss, 2002)

Figure 56. Global buckling at the crown of plastic pipe (UDOT, 2008)

Figure 57. Pipe collapse (UDOT, 2008)

Research by Bishop (1981) showed that the final (long-term) deflection of buried PVC pipe occurs some time after the long-term load on the pipe has been reached. In the study, four PVC pipes were buried in an embankment under 22 ft of soil compacted to different densities, i.e., 86%, 88%, 90% and 93% standard Proctor, and monitored for deflection over the period of over five years. The deflection that initially occurred due to the soil consolidation in the pipe zone, reached the equilibrium (the first one) approximately 100 days after the installation (85% of it was the initial deflection that occurred immediately after completing the installation). The deflection again started to increase after the groundwater level rose above the pipe level (150 days after the installation) and continued to increase until reaching the second equilibrium (the final one). Seasonal groundwater level changes in subsequent years did not affect the deflection readings. The research showed that deflection depends primarily on the soil density in the pipe zone: 1) the higher the density the smaller the final deflection, and 2) the higher the density the shorter the time to equilibrium. Pipe material properties (e.g., modulus of elasticity, but not ring stiffness) scarcely affect the deflection: a PVC pipe has similar time-deflection response as a steel pipe of the same stiffness. The basic conclusions of this study can be extended to other plastic pipes as long as they are not prone to failures due to stress cracking.

Moore and Hu (1995) used viscoelastic analysis of profiled HDPE pipes and the soil surrounding them to further investigate time dependent thermoplastic pipe response. This work demonstrates that since the flexural pipe response (ovaling deflection) is dominated by the soil stiffness, time dependent ovaling in buried HDPE pipes largely results from time dependent soil response. Hoop compression, which also develops in profiled HDPE pipes, is partly resisted by the soil and partly by the pipe, so that time dependent behavior of both HDPE and soil contribute to overall contraction of the pipe circumference over time. HDPE response with time primarily results in stress relaxation, and since time dependent behavior is largely logarithmic with time, there is little change beyond 12 months.

Selig et al. (1994) developed a test to evaluate the capacity of profile wall thermoplastic pipe in hoop

compression, and reported a buckle at the intersection between the crest and the web (see the next paragraph) at a strain level of 3%. Moore and Laidlaw (1997) used the same test and reported buckling in corrugated HDPE pipe with good soil support at strain levels between 2.2% and 4 %. However, they also showed that dramatic decreases in buckling strength occur when soil is prevented from penetrating into the valleys of the corrugation (as a result of soil particle size, or the presence of a filter fabric wrapped around the outside of the structure), so that buckling can occur at strain that is substantially less than 1%. They introduced a critical strain buckling equation for predicting the hoop strain that produces local buckling; the advantage of using critical strain instead of critical stress is that the pipe experiences almost constant strains (it is held in place by the soil), whereas stresses decrease over time due to stress relaxation. Use of critical buckling strain makes it straightforward to use simplified buried pipe deformation theories (e.g., Hoeg, 1968; Moore, 2001) or finite element analyses (e.g., Katona, 1978) to consider the effects of backfill on expected strain, rather than attempting to define empirical 'buckling loads' to include the effects of soil density, where burial depth to induce buckling increases when the pipe is placed in high density backfill (Moser, 2001). The test has not been used for PVC pipe.

PE pipe in gravity applications can usually withstand larger deflection than PVC, which is important since the higher ring bending stiffness of PVC pipes leads to considerably lower deformations in the field. One study at Utah State University conducted in 1994 showed that buckling failure and excessive bending strains do not occur in buried HDPE pipes until the deflections are about 30%, though interpretation of the buried pipe response is complicated by the high levels of lateral pipe confinement that develop in the USU pipe test cell (a cell that has lateral walls moving inwards during the test), the effects of friction on the sides of the cell (substantial shear stresses occur on the steel liner plates with the result that much of the surcharge load is likely transferred to the test structure, rather than passing directly through the soil from top to base), and tests on large diameter pipes have the springlines reasonably close to the sidewalls of the test cell (so that after the pipe begins to experience distress such as local buckling, it is likely that increasing amounts of the load are transferred into the cell side-walls). USU studies seek to determine structural performance (i.e., ring deflection and signs of distress) as function of soil cover depth, type and density (compaction). It was observed that at certain depth of soil, a dimpling pattern on the inside wall occurs as the beginning of localized buckling, and with further increase in soil depth cover, general buckling starts. For example, a 48 in. single thickness liner tested in a soil compacted to a 95% standard Proctor density first showed dimpling in the liner at 2 and 10 o'clock when pipe deflection was 2%. They observed the buckling to be different to "classic buckling"., differences explained by Selig et al. (1994) as local buckling in the pipe liner rather than global buckling.

Localized buckling occurs because the thin elements of thermoplastic profile wall pipes carry stresses largely in compression (Figure 58) and are susceptible to instability in compression. Moore and Hu (1995) explained the three dimensional response of a lined-corrugated thermoplastic pipe, and the effects of wall geometry on the strains that develop (circumferential as well as axial and radial). Further work on other profiles has been reported by Dhar and Moore (2004; 2006).

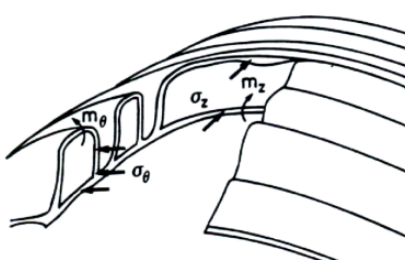


Figure 58. Schematic showing circumferential stress σ_θ and axial stress σ_z in the HDPE profile pipe under external radial stress of 5 psi (Moore and Hu, 1995)

A study of a deeply buried corrugated HDPE test culvert for PennDOT was conducted by Selig (1995). This examined both lined and unlined corrugated HDPE pipes under a deep embankment. Buckling was only observed in the lined corrugated pipe, where it occurred in the liner, and was typically observed in the bottom part of the pipe, between the springline and the invert. The buckle waves appeared in a consistent pattern (Figure 59), as seen in the laboratory (Selig et al., 1994; Moore and Laidlaw, 1997). The study also reported how the extent and degree of buckling increased throughout the six years of field investigation.

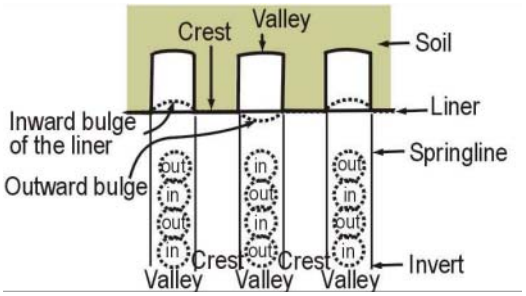


Figure 59. Liner buckling in lined HDPE pipe. Terminology is kept as in the original reference. (Redrawn from Selig, 1995)

NCHRP Report 438 (McGrath and Sagan, 2000) examined the behavior of HDPE and PVC pipes when loaded in hoop compression, and determined that the tested PVC pipe was generally stable and not controlled by local buckling, while in HDPE pipes, local buckling precluded normal ring compression as the mode of failure.

Dhar and Moore (2001) performed tests on large diameter profiled PE pipes to study the development of local buckling under biaxial and asymmetric loading conditions. Observations of local buckling in the internal liners of the pipe products were examined and compared to the results of the stability assessment using the conventional equation for buckling in stiffened plate structures. It was determined that strain levels are controlled by the three-dimensional bending within the pipe profile, and that the modified Bryan equation is a useful design tool for quantifying the liner stability.

NCHRP Report #631(McGrath et al., 2009) includes detailed calculation procedures to calculate critical strain capacity of profiled thermoplastic pipes (strains that induce local buckling), a new ‘stub compression’ test method to determine local buckling capacity using a conventional uniaxial test machine, and new provisions incorporated by AASHTO to control local buckling in thermoplastic pipes. This project also produced a modified deflection equation that includes the effects of hoop compression, and a new technique to include construction effects on the strains that develop in thermoplastic culverts (see Brachman et al., 2008 for the experimental basis).

AASHTO specifications for both PE and PVC pipes require that the pipe must survive a flattening test to at least 20% deflection without buckling, cracking or loss of load-carrying capacity. The PVC pipe must be able to withstand 60% deflection without cracking, however the pipe may buckle or loose load-carrying capacity. In installed pipes, both ASTM D 2412 and AASHTO Section 30 (AASHTO, 2002) limit the allowable initial deflection of the pipe to less than 5% of the actual inside diameter.

2.1.4.4 Oxidative Degradation (Aging)

While plastics are immune to corrosion, they are subject to oxidative degradation due to aging (polyethylene in particular), which over time weakens the pipes and promotes its failure. The only other types of aging mechanisms that may limit the durability of plastic pipes (Mruk, 1990) are photo and thermal degradation (these processes require a long-term exposure to sunlight or sufficient heat and are not real concerns in buried culvert pipes) and slow crack growth under tensile stressing (already discussed in section 0).

Polyethylene resin manufacturers normally add antioxidants (stabilizers) to improve the resistance of PE to oxidative degeneration (oxygen first attacks and the antioxidants, and once they are completely depleted, additional oxygen begins to attack the polymer). Hsuan and Koerner (1998) described the function of antioxidants, the types of antioxidants and the depletion mechanisms. The oxidation of stabilized polyethylene occurs in stages: 1) depletion of antioxidants; 2) induction period (extremely slow, often at immeasurable rate; the polymer reacts with oxygen forming hydroperoxide ROOH, however the amount of ROOH is small and it does not decompose into free radicals), 3) acceleration period (rapid; substantial amount of free radicals is produced leading to cross-linking or chain scission in the polymer; melt index and mechanical properties start changing), and 4) deceleration period (once again slow). The change in melting index is related to the molecular weight of polymer (cross-linking reactions produce an increase in molecular weight and chain scission produce a decrease). The melt down index test, ASTM D1238, assesses the molecular weight of the polymer and can be used as an indicator of oxidation. The most important change in mechanical properties is a decrease in tensile break stress and strain, while to a lesser extent, yield stress increases and yield strain decreases. Ultimately, the engineering performance is jeopardized.

NCHRP Report 429 (Hsuan and McGrath, 1999) found a large variation in the amount of antioxidants in the14 commercially available HDPE pipes they evaluated. The oxidative induction time (OIT) value, expressing the amount of antioxidants, ranged from a few minutes to over 40 minutes (longer OIT correlates to more pipe resistance to oxidation). This indicated that there considerable variability in the resins used in the manufacturing of different corrugated HDPE pipes.

The evaluation of antioxidants in corrugated HDPE pipes is currently not required in the AASHTO M294 specification (except for the cell class defined in ASTM D 3350), but it is often included in specifications by some DOTs. For example, FDOT adopted a comprehensive test protocol specification (Hsuan and McGrath, 2005) to determine the expected service life of corrugated HDPE drainage pipe. This new specification utilizes Oxidation Induction Time (OIT) testing to measure the product durability (FDOT, 2008b).

2.1.4.5 Local Damages

Various types of local damage may occur to the pipe during pipe production, during construction, or under pipe service. This includes voids (cavities inside the pipe wall), blisters (visible voids near the pipe surface), delaminations (debonding of the layers of composite pipes or detachment of pipe liner from the host pipe), as well as dents (scratched or carved surface depressions), pipe wall tears and punctures.

2.1.4.6 Defective Joints

Defective joints in plastic culverts typically involve open joints (Figure 60, Figure 61) with adjacent pipe sections misaligned (i.e., pipe sections are not lined up relative to each other, exhibiting a disjuncture) or they remain aligned but separate (displaced longitudinally). With partially open joints, one pipe section develops more ovality than the other and a drop in the pipe crown is visible (Figure 61). Joint defects can be caused by improper installation, undermining, uneven settlement of fill, or soil infiltration into the culvert through an opened joint).

Figure 60. Separated joint (UDOT, 2008)

Figure 61. Partially open joint with drop (2 in.) in the pipe crown (UDOT, 2008)

2.1.4.7 Sags/Heaving

Thermoplastic culverts can also have sags or heaving due to soil settlement and poor backfill support. Minor sags or heaving are generally not a significant problem in corrugated metal structures unless it causes shape or joint problems. Sags/heaving could be caused by improper installation, undermining, and uneven settlement of fill.

2.1.4.8 Undermining and Bouyancy Problems

Undermining is the loss of soil structural support for the culvert and is manifested as voids in the soil below and around the culvert due to piping, water exfiltration, and backfill infiltration into culvert. In high groundwater conditions, lightweight plastic culverts are susceptible to buoyancy problems (Figure 62) and can appear on the ground surface (Figure 63, Figure 64) (UDOT, 2008).

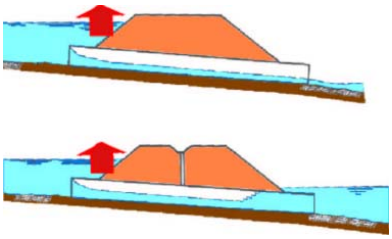


Figure 62. Buoyancy dangers with lightweight plastic pipe (UDOT, 2008)

Figure 63. Example of buoyancy problems (UDOT, 2008)

Figure 64. Example of buoyancy problems (UDOT, 2008)

2.1.4.9 Summary of Defects in Thermoplastic Culvert Pipes

Defect types in plastic culvert pipes and their causes are summarized in Table 6.

Table 6. Culvert distress types in thermoplastic culverts

Defect Type	Description	Cause
Cracking	Mostly circumferential cracks, sometimes longitudinal, elliptical (around local buckles)	Slow crack growth (SCG) or rapid crack propagation (RCP) due to longitudinal bending within the profile, high residual stresses after manufacture, local buckling, or improper installation
Deflection, buckling (localized, global)	Ovality, dimpling and ring buckling, wall crushing at springlines, reversal of curvature at crown or invert	Vertical (e.g., earth) loading, compressive stresses in thin elements of profile walls
Oxidative degradation	Change in mechanical properties (tensile break stress and strain) over time	Aging over time due to exposure to oxygen
Defective joints	Separated, misaligned joints	Improper installation, undermining, uneven settlement of fill, or soil erosion due to water infiltration into the culvert through an opened joint
Sags/heaving	Vertical misalignment between joints	Improper installation, undermining, uneven settlement of fill
Undermining	Voids in the soil below and around the culvert	Piping, water exfiltration, backfill infiltration into culvert
Buoyancy	Decrease in depth of soil cover, pipe emerging on the surface	Upward force on a lightweight pipe in high groundwater conditions

2.2. HYDRAULIC CAPACITY OF CULVERTS

2.2.1. OBJECTIVE AND NOTABLE REFERENCES

This section reviews the basics of culvert hydraulics, which is generally concerned with the culvert hydraulic performance under a wide range of headwater and tailwater elevations for different culvert geometries, materials and inlet configurations. Occurrence of roadway topping is of special interest, as well as the effect of rehabilitation measures on the hydraulic performance of culverts.

The FHWA's ***Hydraulic Design of Highway Culverts*** (Norman et al., 2005) is a comprehensive culvert hydraulic design publication that combines information previously contained in Hydraulic Engineering Circulars (HEC) No. 5, No. 10, and No. 13 (Herr and Bossy, 1965; Herr and Bossy, 1972; Harrison et al., 1972) with hydrologic, storage routing, and special culvert design information. Culvert design methods are presented for both conventional culverts and culverts with inlet improvements. Inlet control, outlet control, and critical depth design charts are included for a variety of culverts sizes, shapes, and materials. Dimensionless culvert design charts are provided for the design of culverts lacking conventional design nomographs and charts. The appendices of the publication contain the equations and methodology used to construct the design charts, information regarding the hydraulic resistance of culverts, and methods for optimizing culvert design using performance curves and inlet depression. Calculation forms are provided for several design methodologies. Some basic concepts related to hydraulics of culverts are outlined in this chapter.

The FHWA's ***Highway Hydrology*** (McCuen et al., 2002) describes the hydrologic methods used for the design of culverts.

2.2.2. FLOW CONDITIONS

Culvert flow conditions can be classified as either:

- Full flow, usually a pressure flow (Figure 65), although a special condition may exist where a pipe flows full with no pressure, a.k.a. just full flow.
- Partly full flow, free surface flow, a.k.a. open channel flow (Figure 66), which depending on Froude number (F_r) can be subcritical (slow, tranquil flow) for $F_r<1$, critical for $F_r=1$ or supercritical (fast, rapid flow) for $F_r>1$.

Figure 65. Full flow (Najafi et al., 2008a)

Figure 66. Partly full flow (U.S. Fish and Wildlife Service, 2008)

2.2.3. TYPES OF Flow Control

The hydraulic operation of culverts is rather complex. Two types of flow control in culverts can be distinguished:

- Inlet control - The hydraulic capacity of a culvert is controlled at the culvert entrance. The culvert barrel is capable of conveying more flow than the inlet will accept.
- Outlet control - The hydraulic capacity of a culvert is controlled at the culvert barrel exit or further downstream. The inlet opening is capable of accepting more flow than the culvert barrel is capable of conveying.

For each type of flow control, different flow conditions can develop (Figure 67). Examples of inlet control shown in Figure 67 were developed for flow that is partly full for the entire length of the barrel or part of it. Critical depth forms just downstream of the culvert entrance and supercritical flow is created downstream past it. In examples with unsubmerged outlet (A, C), depth of flow approaches normal depth near the end of the culvert, whereas in examples with submerged outlet (B, D), hydraulic jump forms in the culvert barrel. Even submergence of both ends (D) does not assure full flow in the barrel.

In examples of outlet control in Figure 67, culverts flow is full and under pressure for the entire length of the barrel (A, B, C) or part of it (D), or the flow is partly full with subcritical flow (E). Condition B has shallow headwater (B) resulting in the inlet crown exposed and the flow contracting in the barrel. Condition D requires very high headwater to maintain full flow with low tailwater and it occurs rarely.

For most situations in mountain streams, inlet control prevails because culverts are often short and slopes relatively steep, so the length and barrel slope can be ignored (Douglas, 1974).

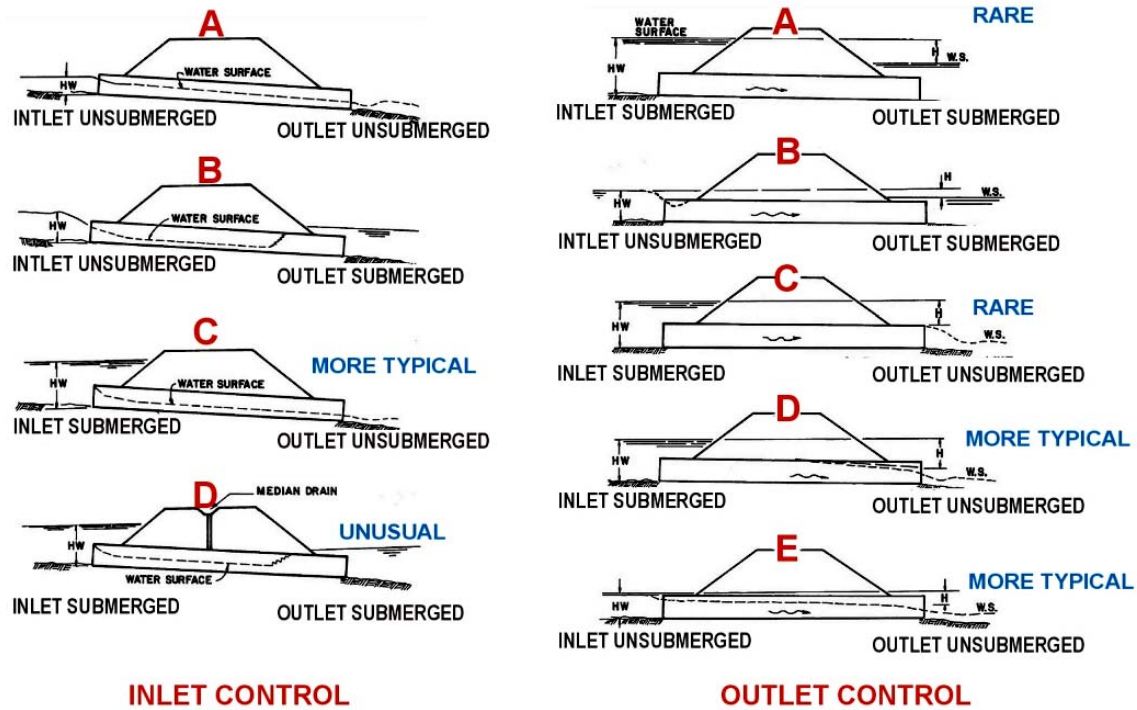


Figure 67. Some examples of flow conditions in culverts operating under inlet control and outlet control (Redrawn from Norman et al. 2005)

2.2.4. FLOW CAPACITY

For each type of flow control, the flow capacity of a culvert depends on different factors (Table 7). The flow capacity of culverts operating under inlet control is determined mainly by inlet geometry, pipe barrel cross-sectional area and headwater depth. The flow capacity of culverts operating in outlet control is governed primarily by the tailwater depth, pipe slope, roughness and length, and is normally calculated using Manning's equation.

Table 7. Factors influencing culvert performance for different types of flow control (Norman et al., 2005)

Inlet Control	Outlet Control	
Headwater elevation	Headwater elevation	Barrel roughness (Manning's value)
Inlet area	Inlet area	Barrel cross-sectional area
Inlet edge configuration	Inlet edge configuration	Barrel shape
Inlet geomerty	Inlet shape	Barrel length
Barrel slope (to a small degree)	Barrel slope	Tailwater elevation

Performance curves. Performance curves plot headwater depth vs. flow rate. Inlet control performance is defined by three regions of flow: unsubmerged (low headwater conditions, the culvert inlet operates as a weir), submerged (high headwater conditions, the culvert inlet operates as an orifice), and a transition between these two conditions (Figure 68). Appendix A of Norman et al. (2005) contains the equations for unsubmerged and submerged conditions based on model test data. The transition flow zone is poorly defined and is approximated by plotting unsubmerged and submerged flow equations, and connecting both curves with a line tangent (Figure 68).

Outlet control flow conditions can be calculated based on energy balance. Norman et al. (2005) contains the equations for full barrel flow applicable for outlet conditions A, B, and C in Figure 67, as well as methodology and nomograms developed for analyzing partly full flow for outlet conditions D and E.

Culvert performance curves contain plots for both inlet and outlet flow controls (Figure 69). The result that gives the lower culvert capacity (or the higher upstream water level) indicates the type of flow that controls the operation of the culvert. Performance curves show the consequences of higher flow rates at the site and the benefits of inlet improvements.

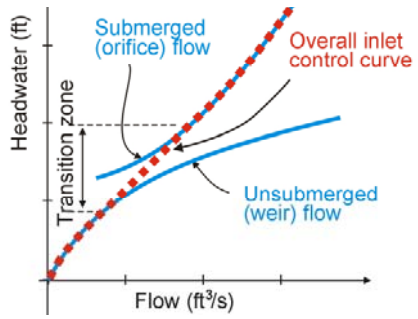


Figure 68. Inlet control curves (Redrawn from Norman et al., 2005)

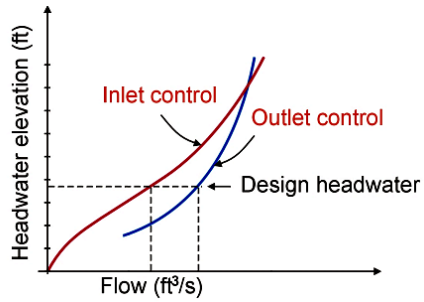


Figure 69. Culvert performance curve (Redrawn from Norman et al., 2005)

Research related to calculating culvert hydraulic performance. Iowa State University developed discharge formulas for various culverts based on 3,301 experiments on the flow of water through short conduits such as pipe and box culverts and sluiceways under levees (Yarnell et al., 1926).

Pennsylvania State College developed a nomographic scale for calculating the hydraulics of culverts based on studies conducted between 1939 and 1942 (Mavis, 1943).

The FHWA's ***Hydraulic Charts for the Selection of Highway Culverts*** (Herr and Bossy, 1965) contains a series of performance curves and nomographs for calculation of culvert performance under both inlet and outlet control for commonly used entrance configurations and culvert materials.

Ead et al. (2000) investigated the velocity field in turbulent open-channel flow in a circular corrugated pipe at different slopes and discharges. The Manning coefficient *n* was found to be equal to 0.023.

Charbeneau et al. (2006) developed a two-parameter model describing the hydraulic performance of highway culverts, which can accurately represent the FHWA performance curves.

Hotchkiss et al. (2008) compared several computer programs available to analyze culvert hydraulics, including HY-8, FishXing, Broken-back Culvert Analysis Program (BCAP), Hydraflow Express, CulvertMster, Culvert, and Hydrologic Engineering Center River Analysis System (HEC-RAS). The flow controls, headwater depths, and outlet velocities obtained with different programs were compared with values obtained through calculations based on best practice outlined in Norman et al. (2005). Several limitations were identified. BCAP has limited ability to analyze straight barrel culverts operating under outlet control with high tailwater, CulvertMaster reports outlet control for some low discharges as a surrogate for what is referred to as "entrance control," Culvert misidentifies the location of hydraulic control for lower discharges and overestimates headwater elevations, and HydraFlow Express incorrectly uses critical depth at the outlet for inlet control calculations. For the test cases used, HY-8, HEC-RAS and FishXing most consistently agreed with accepted empirical results. It was recommended that program providers improve their products to better replicate the hydraulic conditions simulated in this paper and to extend program capabilities to include environmentally sensitive design considerations such as fish passage.

2.2.5. ROADWAY OVERTOPPING

When the headwater rises to the elevation of the roadway, overtopping of the roadways occurs (Figure 70). The flow is similar to the flow over a broad crested weir. The flow across the roadway (Q_0) is calculated from the water depth above the roadway (HWr).

The overall culvert performance curve with roadway topping (Figure 71) depicts the sum of the flow through the culvert and the flow across the roadway. Using this curve, it is easy to determine the headwater elevation for any flow rate, or to visualize the performance of the culvert over a range of flow rates.

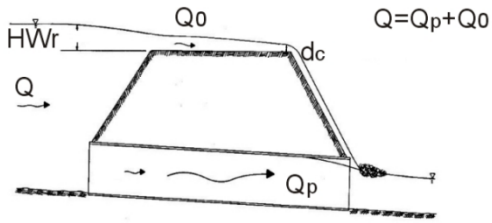


Figure 70. Roadway overtopping (Redrawn from Norman et al., 2005)

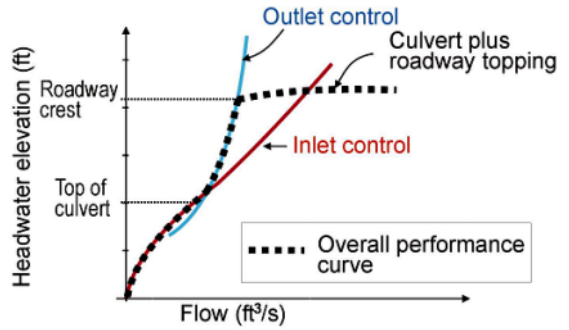


Figure 71. Culvert performance curve with roadway topping (Redrawn from Norman et al., 2005)

2.2.6. ECONOMICS OF Culvert Hydraulic Design

Hydrology. Hydrologic analysis involves estimation of peak flows (peak discharge) of various return intervals from surface runoff, which are used as design flow rates in the culvert design

The FHWA's ***Highway Hydrology*** (McCuen et al., 2002) describes hydrologic methods important for the design of culverts. The rate of runoff passing through a given point along the stream during or after a rainfall event depends on climate and watershed characteristics.

Flood frequency in hydraulic design. During the service life of a culvert, a wide spectrum of flows with associated flood probabilities will occur. The design return period should be selected to yield most economical culvert size. The objective is to find the optimum culvert capacity that accommodates expected flows during the service life of culvert while minimizing the associated costs both in the construction and the operation phases. Consideration of storage routing (the attenuation of the flood flow due to the storage volume upstream of the culvert) often reduces the design culvert size.

This relatively simplistic design approach is commonly suitable for minor stream crossings, however a risk analysis based on a benefit cost analysis (BCA) may be needed for large culverts. The BCA compares the benefits (decreased traffic interruption time due to road flooding, eliminated flood damages, increased safety) with the costs (the initial construction of culvert and road embankment) for different culvert capacities (Figure 72). (Norman et al., 2005)

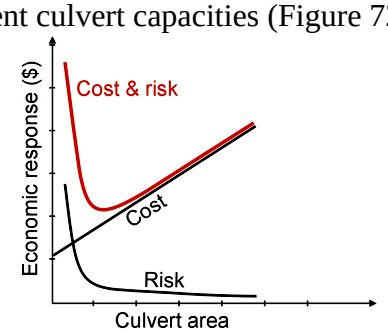


Figure 72. Risk analysis benefit versus cost curve (Redrawn from Norman et al., 2005)

Risk analysis in decision-making process. The FHWA's ***Hydraulic Engineering Circular (HEC) No.17*** (Corry et al., 1981) provides guidance in the application of the least total expected cost (LTEC) design decision-making process, which is applicable in the design evaluation of culverts (the emphasis in this manual is on bridge crossings). An essential part of the LTEC design concept is risk analysis. Risk analysis provides the means for analyzing the losses incurred for the various design strategies due to possible states of natures (flood events). All quantifiable losses are included in the risk analysis. The product of the risk analysis is the annual economic risk associated with each design strategy. The sum of the annual economic risk and the annual capital costs (i.e., the total construction costs multiplied by a capital recovery factor) yields the total expected cost (TEC) for each design strategy. Comparison of the TEC's for all design strategies allows the designer to select the LTEC or optimum design.

An example in Appendix B of Corry et al. (1981) illustrates the application of risk analysis to the design of a culvert. A circular culvert 100 ft long should be designed under a two-lane highway, with average traffic 3,000 vehicles per day, discount rate 7.125% and culvert's useful life 35 years. Six floods were used in the analysis (return interval 5, 10, 20, 40, 80, 160 years) and four alternative culvert diameters (48 in., 54 in., 60 in., and 66 in.). The economic losses (traffic interruption, backwater and damage to the embankment) were calculated, as well as annual capital and maintenance costs. The annular risk cost for alternative designs shows that 48 in. is the LTEC design if assuming the culvert does not fail under any flood condition, and it is 54 in. if assuming the culvert fails when embankment losses are greater than 50%. Embankment loss is an estimated volume of embankment that is lost due to erosion during the roadway overtopping; it depends on the duration of overtopping, in hours, and the height of overtopping, in feet (see Figure 73). These estimates are generally based on experience accumulated in some highway agencies; the available datasets are typically incomplete, especially for larger overtopping heights. In this example, the 50% embankment loss was estimated to occur only if a 48 in. culvert was constructed and floods were 170 cfs or greater, which corresponds to a 20-year return interval or longer. Such embankment loss would yield culvert damage and result in an additional cost in the computation of the annual risk costs due to the cost associated with the culvert replacement.

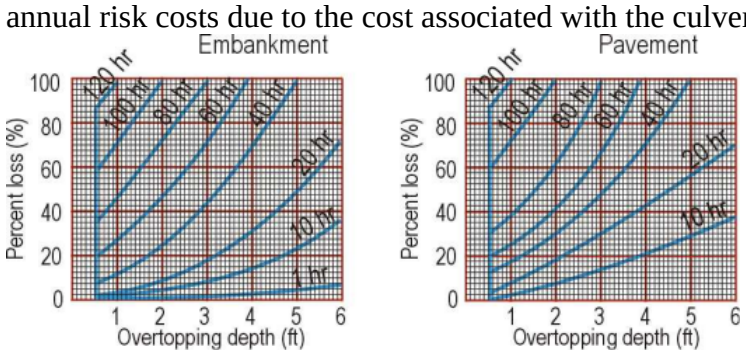


Figure 73. Embankment and pavement losses (Redrawn from Corry et al., 1981)

Tung and Bao (1990) further investigated risk-based design procedures applied to culvert design, in particular the sensitivity of the optimal design parameters to (1) the hydrologic parameter uncertainty, (2) the length of stream-flow records, and (3) the distribution model of flood flow. Uncertainties in hydraulic, structural, and economic aspects were not considered.

2.2.7. EFFECT OF CULVERT REHABILITATION ON CULVERT HYDRAULIC PERFORMANCE

Most culvert rehabilitation methods involve insertion of some type of liner and thus reduce the inside diameter of culverts after the rehabilitation is completed. On the other hand, new materials typically have smoother surface that is favorable for increasing flow capacity. This section reviews research that examined the effect of rehabilitation on the hydraulic performance of rehabilitated culverts.

Newton (1999) showed that flow capacity of corrugated metal culverts sliplined with HDPE pipe was maintained or increased despite diameter reduction (Table 8). The Manning coefficient for HDPE pipe was 0.009.

Table 8. Corrugated metal culvert flow capacity when sliplined with "Culvert Renew"® (Newton, 1999)

Original Pipe ID (CMP)	New Pipe ID (sliplined)	% of Original CMP Flow	Original Pipe ID (CMP)	New Pipe ID (sliplined)	% of Original CMP Flow
12 in.	10 in.	164%	30 in.	24 in.	147%
15 in.	12 in.	147%	33 in.	27 in.	156%
18 in.	15 in.	164%	36 in.	30 in.	156%
21 in.	15 in.	109%	48 in.	36 in.	124%
24 in.	18 in.	124%	54 in.	40 in.	120%
27 in.	21 in.	136%	60 in.	42 in.	103%

Kozman (2006) developed a simplified approach to estimate the hydraulic capacity of CMP culverts relined with closed-fit liners. Several man-entry culverts (galvanized and bituminous-coated CMP) in OH and CA that had been exposed to heavy corrosion and abrasion over the years were relined with CIPP or deformed/reformed HDPE. The culverts were inspected 2.5 years after relining and measurements were taken of the inside diameter, interior roughness and upstream end geometry. Actual thicknesses of the installed liners measured in the field were between 0.63 in. and 1.20 in. for CIPP and 0.63 in. and 0.75 in. for HDPE. Estimations were made for entrance loss coefficients, Manning coefficient, and flow capacity for each culvert inspected. Manning coefficient for CIPP segments was 0.0138-0.0167 before lining and 0.0109-0.0135 after lining, and for HDPE segments 0.0140 before lining and 0.0082-0.0086 after lining. Percentage flow capacity maintained was 93-139% for CIPP and 135-142% for HDPE. The research demonstrated that close-fit liners can be used to restore or increase the flow capacity of deteriorated culverts. Finished interior roughness of close-fit liners (e.g., HDPE, CIPP) is dependent on interior roughness and condition of the host pipe, and liner material properties, thickness and installation procedures.

Wallace et al. (2007) investigated hydraulic performance of CIP relined corrugated metal culverts on twin CMP culverts near South Haven, MI (originally 7 ft 8 in.×5 ft 5 in. and 228 ft long). Field measurements were taken and hydraulic calculations made for the 100-yr peak flow conditions. CIPP lining reduced the cross sectional area of the CMP culvert from 33.0 ft^2 to 28.9 ft^2. Manning coefficient dropped from 0.034 to 0.013. The relative importance of these two outcomes of the lining process was explored by detailed comparison of the energy losses that occur under outlet control when the barrel is full (the design flow the hydraulics of these culverts are governed by outlet control). It was found that the improvement in total energy loss due to reduced surface roughness requires certain minimal length of pipe to offset the negative impact of cross-area reduction. In this particular case it was calculated to be 38 ft.

Table 9. Problems related to hydraulic capacity of culverts

Problem	Cause
Overtopping of roadway Failure of culvert and the roadway above Flooding of adjacent properties	Unexpected headwater depth due to inadequate hydraulic capacity
Damage to embankment and appurtenances Flooding of downstream areas	Erosion and abrasion
Local scour (a hole at the outlet)	High-velocity flow at the downstream culvert outlet
Channel degradation (head cutting)	High-velocity flow within the culvert

2.3. CONVENTIONAL INSPECTION METHODS OF MAN-ENTRY CULVERTS

2.3.1. INTRODUCTION

This section reviews methods for inspecting culverts that are feasible for person entry. Culverts as small as 30 in. in diameter can be entered for physical inspection although diameters 36 in. and above are more commonly entered. Even with large diameter culverts, access may be limited by one or more factors: accessibility of the inlet and the outlet, water depth, velocity of the flow, debris accumulation, structural condition, presence of toxic gases, or other hazardous conditions. Whenever attainable, however, physically entering the culvert is the simplest method for locating, observing and measuring various defects.

2.3.2. VISUAL INSPECTION

Many defects in man-entry culverts can be detected by visual inspection as defects on the inner surface of the culvert are visible to the naked eye. Types of defects to look for when inspecting the culvert barrel depend upon the type of culvert being inspected. In general, concrete culverts are inspected for cracks, spalls and other surface defects, and defective joints. Corrugated metal culvert barrels are inspected for cross-sectional shape deformities and barrel defects such as joint defects, seam defects, plate buckling, lateral shifting, missing or loose bolts, corrosion, excessive abrasion, and localized construction damage. Thermoplastic culvert barrels are inspected for shape distortion, buckling, dents and tears, and defective joints. All types are inspected for presence of sediments and debris.

In addition to visual observation, direct measurements of diameter and alignment provide important information for completing the condition assessment.

Diameter measurements determine shape distortion of pipes. A deflection meter can be used (Figure 74), which is comprised from a 1.5 in. white PVC tube and a 1 in. PVC yellow scaled tube that can slip out of and into the white tube (Blackwell and Yin, 2002).

Figure 74. Measuring horizontal diameter with deflection meter (Blackwell and Yin, 2002)

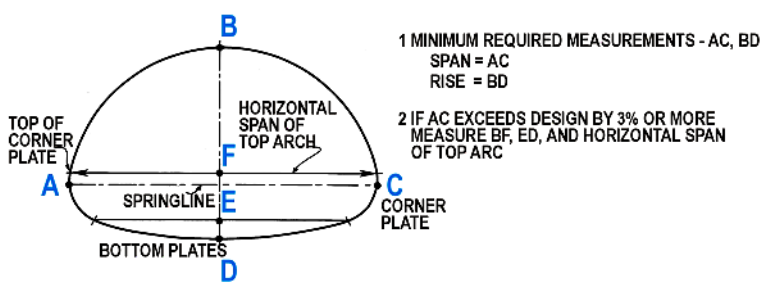


Figure 75. Shape inspection of structural plate pipe arch (Redrawn from Ballinger and Drake, 1995)

Ballinger and Drake (1995) specified necessary measurements in shape inspections. For example, in a structural plate pipe arch, at minimum the horizontal span at the base of the arch and the rise must be measured, and depending on the obtained results, additional measurements may be needed (Figure 75). Diameter measurements are taken at the springline of pipe at short distances apart, for instance every 5 ft along the length of the pipe. At each measurement location vertical, horizontal, and each of 45° angle diameters are measured (Nelson and Krauss, 2002).

Alignment measurements determine pipe misalignment or uniformity of curvature. For this purpose, a laser set up at the end of the pipe (either inlet or outlet) can measure the distance from the pipe crown to the level every 5 ft along the length of pipe (Nelson and Krauss, 2002).

Sounding. Visual inspection can often be aided by simple tools that provide additional information. One such tool is the sounding hammer (FHWA, 2006) used in concrete culverts to detect delaminated areas. A delaminated area will have a distinctive hollow "clacking" sound when tapped with a hammer. A hammer hitting sound concrete will result in a solid "pinging" type sound. The same method can be used in metal culverts, e.g., for checking the bolts on the longitudinal seams in steel culverts (by tapping the nuts with the hammer). For aluminum structural plate, the bolts are checked with a torque wrench. A geologist's pick hammer can be used to scrape off heavy deposits of rust and scale. The hammer can then be used to locate areas of corrosion by striking the culvert walls. The walls will deform or the hammer will break through the culvert wall if severe corrosion exists.

2.3.3. DESTRUCTIVE INSPECTION METHODS

Core sampling. Drilling and probing are used to locate defects behind or outside the culvert. A series of holes are drilled circumferentially at predetermined locations along the length of the culvert. The holes are probed with various tools to determine the thickness of the liner, the density of the material behind the liner and/or the presence of a void behind the liner. Data collected is location specific.

Thiel et al. (2008) reported collecting of core samples as part of pre-rehabilitation inspection of two tunnels in Tennessee. Ground penetrating radar (GPR) technology was employed to obtain the necessary information (see 2.4.6.3 for more details) and core sampling was performed to validate the GPR survey results. A total of eight cores were taken through pavement down to the bedrock depth; core lengths were then directly measured to determine the depth to bedrock at each location drilled. The results were made available for comparison purposes with the planned radar survey findings. Additional details are given in section 2.4.6.3.

2.4. NONDESTRUCTIVE INSPECTION (NDI) OF CULVERTS

2.4.1. INTRODUCTION

This section reviews nondestructive inspection (NDI) methods that are applicable to non-man-entry culvert pipes, as well as non-conventional methods applicable to man-entry culvert pipes. A web site, *NDT Resource Center*, http://www.ndt-ed.org, has been identified as a comprehensive source of information and materials for NDT and NDE technical education. The site hosted by the Center for NDE, Iowa State University, was created with the funding from the National Science Foundation (NSF).

Nondestructive inspection (NDI), also called nondestructive testing (NDT), includes methods that can detect internal or external imperfections of an object or material, determine composition, mechanical properties, or measure geometric characteristics (SJSU, 2008). A large number of NDT techniques have been developed for different purposes - over 50 according to the International Atomic Energy Agency (IAEA, 2006), but only some are suitable for investigation of culverts.

NDT methods can generally be divided into visual (methods that provide images of the inside surface of a pipe wall) and non-visual (methods capable of detecting defects or measuring physical properties inside the pipe wall, on the outside surface or in the soil envelope surrounding the pipe).

Optical methods (CCTV, scanning and laser profiling) and ground penetrating radar (GPR) applied from the ground surface are fairly well established methods in current practice, while other methods described in this chapter are emerging and have been so far used mainly in studies that evaluate their applicability.

2.4.2. OPTICAL METHODS

2.4.2.1 CCTV

CCTV is a common method of inspecting non man-entry culverts. A camera is mounted on a wheeled platform capable of traveling along the culvert, thus allowing examination and evaluation of the entire length of the culvert barrel. The camera is used in conjunction with a video monitor and video recorder or other recording devices. The inner surface of the pipe is videoed during the inspection and recorded images are reviewed during or after the inspection.

CCTV pipe inspection systems can be analog or digital. Digital systems have advantages compared to analog systems (Redzone Robotics, 2008b):

- Analog signal degrades a little bit each time it is reproduced. Digital signal does not degrade over time, distance or with copying.
- Once it leaves the camera, analog signal is susceptible to noise and interference while being transmitted over cable. Digital signal filters out noise and interference with greater ease, and transmits over long distance without loss of image quality.
- Digital technology allows digital enhancement (e.g., adjustment of image contrast, sharpness, etc), digital manipulation (e.g., images can be digitally compressed), and increases portability (e.g., images can be sent over internet)
- Analog technology requires skilled CCTV operator who pan, tilts and zooms on site. Digital technology allows off-line analysis of recorded image, and a rapid off-site review by qualified personnel.

Overall, CCTV inspection can detect various defects, e.g., cracks, spalling, dents, bulges, cracks, pinholes, etc, as well as sags, open joints, misalignment, shape distortion, etc. The inspection is limited to defects above the waterline and is preferably conducted when the culvert is empty.

The camera may be limited to front viewing or have a “pan and tilt” option (cameras specifically designed to provide a close-up view of pipe walls and a radial rotation viewing of up to 360° (Figure 76). The pan-and-view cameras have overcome some limitations of front-viewing cameras (Figure 77), but operator’s skill remains a critical variable because the camera has to be stopped during the inspection for pan-and-tilting.

Figure 76. Pan-and-tilt camera

Figure 77. Images obtained with a pan-and-tilt camera: front view and two side close-up views (Courtesy of CUES Inc)

Image reviewing is typically carried out “manually”: an operator (or engineer) detects, classifies and rates the severity of defects against the documented criteria. Inspection results and conclusions highly depend on the experience, capability and concentration of the operator (e.g., Wirahadikusumah et al., 1998). Generally, results of CCTV inspection are recognized to have a lack of consistency and reliability, especially when the objective of the inspection is to track the process of deterioration for planning of the proactive management. Misjudgment as to the defects severity is easily made, and can impact the decision about the type of action that follows the inspection, i.e. whether the pipe would be rehabilitated and the timing of such action (assigned priority) (Makar, 1999).

Dirksen and Clemens (2008) outlined the possibilities of using CCTV inspection data for deterioration modeling and discussed the problems encountered. A case study was performed on modeling surface damage caused by corrosion or mechanical action using a Markovian model.

Software for storing and reporting results of CCTV pipe inspection is an important link between pipe inspection cameras and actionable information. Software providers strive to make their applications versatile, flexible, easy to use, and compliant with standardized defect coding systems. One such software, Pipeline Observation System Management (POSM) (pronounced "possum") supports inspections for many types of infrastructure and allows extensive user customization (Figure 78 and Figure 79). The software is certified under the Pipeline Assessment and Certification Program (PACP) Version 4.2, the latest database and testing standard from the National Association of Sewer Service Contractors (NASSCO) (Rulseh, 2008).

Figure 78. POSM interface for data entry (RS Technical Services, 2008)

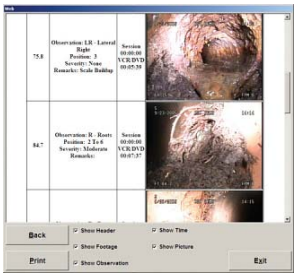


Figure 79. Example of POSM report (RS Technical Services, 2008)

Another software, Granite XP, offers two ways to organize and display pipeline inspections: the classic "project-based" method (note: a project is defined as a set of inspections) and the "asset based" method that allows importing municipal assets (e.g., pipelines, manholes, lateral service connections etc) to be inspected, tracked and managed at the asset level. The "Project Navigator" displays the set of inspections defined by the user (Figure 83), whereas the "Asset Navigator" displays all of the historical inspections of a particular asset (Figure 81). The software contains Microsoft Windows™ layout screens that offer drop-down menus, a fast auto complete feature for quick and easy look-ups, and detailed Help files within the program. The software offers four editions: 1) inspection edition (allows users that perform pipeline inspections in the field to capture, assess and store inspection data), 2) office edition (allows users to manage inspection information and create customized inspection analysis in the form of reports, videos, still pictures, and database files to meet a wide array of data requirements. Capable of running on Enterprise databases such as Oracle 9i and SQL Server), 3) engineering edition (allows users to modify inspections and observations gathered in the field, to review existing data, synchronize inspections, capture images from playback and generate reports) and 4) viewer edition (allows users to review and share information gathered during inspections and generate reports) (CUES, 2009).

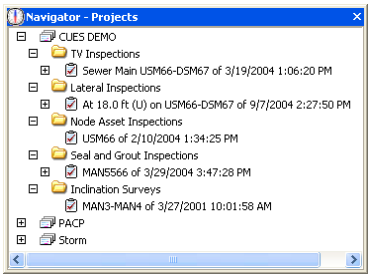


Figure 80. Project Navigator in Granite XP (CUES, 2009)

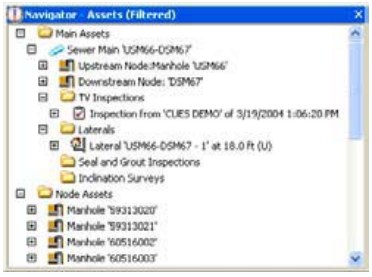


Figure 81. Asset Navigator in Granite XP (CUES, 2009)

Figure 82. User interface for entering field inspection data in Granite XP (CUES, 2009)

Example case study. Jacksonville Naval Air Station, FL, had 12 RCP culverts CCTV inspected prior to relining. The inspection was done with pan and tilt camera on over 2,000 ft of culverts, 6”-30” in diameter, and at depth 3-12 ft (Dayton, 2008a; 2008b).

2.4.2.2 Optical Scanning

Optical scanning is the method in which a probe moving through the pipe acquires a digital, full circumferentially unfolded image of the pipe interior surface.

The CERF (2001) report described one early optical scanning technology for sewer pipes assessment. The scanned image was obtained as follows: (1) a halogen lamp in the probe produced light, which was directed onto a two-piece scanning mirror, angled at approx 90° and rotating 54 times a second; (2) the light was reflected by the mirror on the forward side of the rotary assembly, illuminating a spot on the interior wall of the pipe; (3) the image of the illuminated spot was reflected on the mirror on the other side of the rotary assembly and guided through a lens to the optical sensor, and the image was converted into a digital signal in full color through an analogue/digital converter. The probe collected data for 500 dots in each rotation, thus creating a scanned line (Figure 83). An example of raw image is shown in Figure 84.

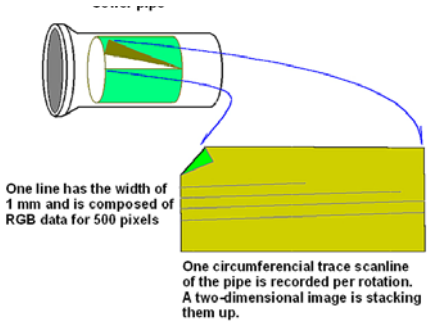


Figure 83. A schematic of how the unfolded image was obtained by the early version of optical scanning (CERF, 2001)

Figure 84. Unfolded “raw image” of pipe interior produced by optical scanning (CERF, 2001)

Figure 85. Model of optical scanner (Knight et al., 2009)

Further development of optical scanning lead to systems equipped with a fish-eye lens, which use a single line of image pixels to construct the unwrapped, circumferential image of the inner surface of the pipe wall. Koo and Ariaratnam (2006) reported the use of one such system in combination with the ground penetrating radar (GPR) for inspection of large diameter PVC lined concrete pipe in Phoenix, AZ. Knight et al. (2009) reviewed advances in digital scanning evaluation camera technology.

Gunn (2008) described another optical scanning system that has a 3D optical scanner mounted on a self-propelled crawler (Figure 86), which is equipped with two fixed, high-resolution digital cameras (one at either end of the cylinder). Each camera has a fish-eye lens that captures a 185° view (a hemisferical image) of the pipe. As the scanner moves through the pipe, it records forward and rear images every 2 in. of travel (a high-resolution digital still image). The images are recorded by computer software, which combines them to make a complete 360° spherical image of pipe wall. Advantages of such 3D optical scanning systems include image sharpness (3,000 lines compared to conventional 500 lines video images) and high speed of inspection (up to 69 ft/min compared to 30 ft/min, which is the NASSCO limitation for CCTV). The system can be used in concrete culverts but some bouncing can be expected in corrugated pipes (personal communication with David Daake, RapidView LLC).

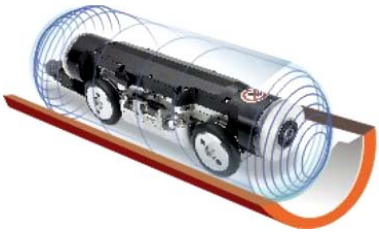


Figure 86. Panoramo 3D Optical Scanner (RapidView, 2008)

Figure 87. Perspective view (Gunn, 2008)

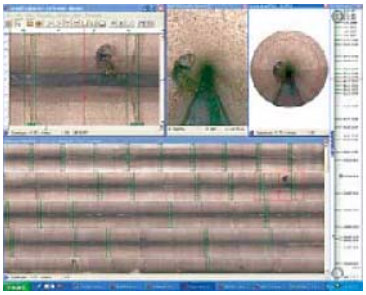


Figure 88. Data display (Gunn, 2008)

Stein and Brauer (2004) had compared the performance of the optical scanning system with a standard CCTV inspection system with respect to picture quality and sharpness.

Kim et al. (2007) described the development of a new digital pipeline scanning system, which uses one lens and one CCD camera to record both front and side views. Side view unwrapping and stitching technology using image process techniques that deliver high resolution image data have been developed. The research would continue with efforts focused on the accuracy and reliability improvement in distance reading.

2.4.2.3 Laser Profiling

2-D laser profiling. Laser profiling is a technology that uses a laser beam to scan the interior of a pipe and determine its ovality, alignment, diameter and capacity, as well as defects such as surface cracks, corrosion (from pipe inside surface), deposits, etc. The term implies 2-D imaging of the pipe's cross-section. 2-D lasers are the most prevalent laser technology used in pipe inspection. The technology is based on structured light (i.e., a light pattern is projected at a known angle onto an object) from a transmitting laser, which is imaged by the detecting camera.

Based on the configuration of the device used to generate the light pattern, 2-D profiling techniques can implement (Duran et al., 2002):

- Single spot scanning method: Individual light spots are projected against the pipe wall employing a rotating mechanism (Figure 89). One commercial profiling tool, for example, takes around 1000 measurements in a 360° scan in 5 seconds (OMC, 2001a). The resolution of the device is 0.25 mm and the diameters of pipes that can be measured with this method range from 9 in. to 60 in.
- Whole circle image method: An optical pattern generator projects a full ring of light onto the wall at once (Figure 90). Optical ring generators are usually an assembly of lenses and conical reflecting mirrors or prisms. Zhang and Zhuang (1998) described two types of optical ring generators (one uses the conical reflection method and the other the method of diffraction grating element).

As the platform moves through the pipe, the connecting of consecutive acquired profiles allows creation of surface map of the inner pipe wall. The choice between the whole-circle-image method and the single-spot-scanning method is a trade-off between accuracy and inspection time. The whole-circle-image method allows faster data acquisition, but has been found to be somewhat less accurate (Duran et al., 2002).

Analysis of the acquired images is based on the positional information provided within the ring shaped pattern in the camera image using the method known as optical triangulation (Figure 91) (OMC, 2001b). The image data analysis is discussed in various publications (e.g., Clarke 1990; Clarke 1995; Duran et al., 2002; Swanbom et al., 2005; Swabom, 2007).

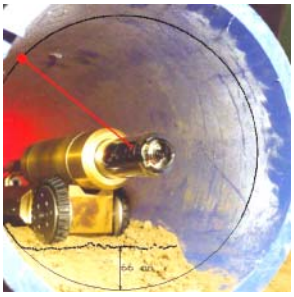


Figure 89. Single spot scanning. Measurement data is super-imposed onto a picture of the profiler in action (OMC, 2001a)

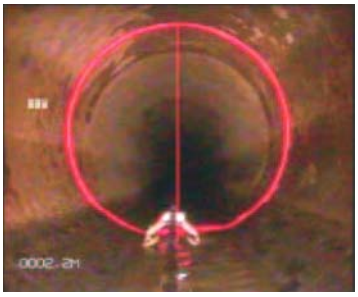


Figure 90. Whole circle image method. The ring of laser light inside a round pipe without roundness defect (Kenter, 2008)

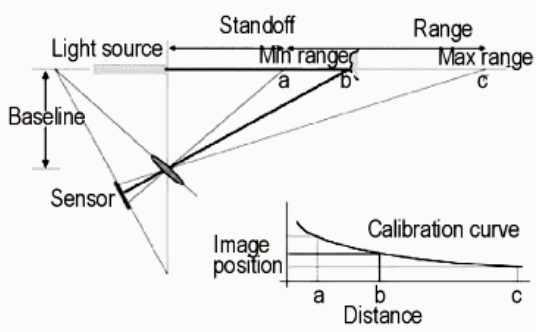


Figure 91. Geometry of optical triangulation (Redrawn from OMC, 2001b)

OMC (2001a) showed the ability of laser profiling to measure size and shape of surface defects by detecting gouges in a plastic pipe (Figure 92).

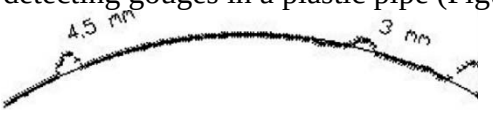


Figure 92. Gouges in a plastic pipe measured by laser profiling (OMC, 2001a)

Bennett and Logan (2005) referenced several case studies in which laser profiling was utilized thus demonstrating capabilities of the method to detect different defects in pipes (i.e., ovality, corrosion, a failed CIP liner). For example, in Louisville, KY, a 36 in. corrugated HDPE stormwater pipe was inspected for ovality one year after the installation. The Ovality Analysis Report (Figure 93) showed major shape variation throughout the entire length of pipe (Figure 94, with maximum ovality of 15.8% and minimum of 5.7%. In addition, cracking of the internal wall was detected along the pipeline (Figure 95).

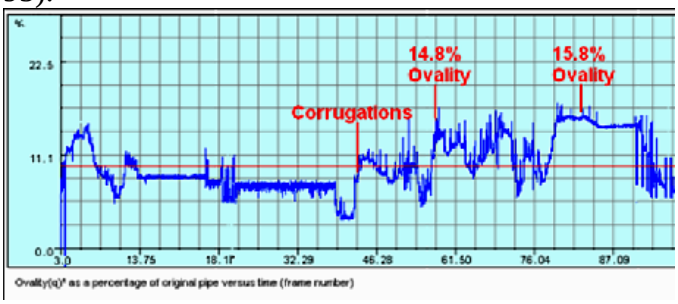


Figure 93. Ovality Analysis Report for a corrugated HDPE pipe (Detail from Bennett and Logan, 2005)

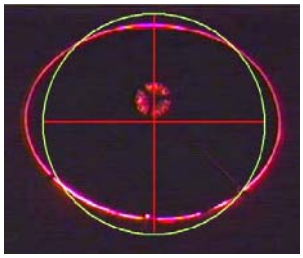


Figure 94. Ovality measured in an HDPE pipe (Bennett and Logan, 2005)

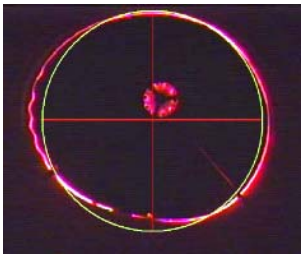


Figure 95. Pipe cracking (Bennett and Logan, 2005)

In Tauranga City, New Zealand (Bennett and Logan, 2005), a 24 in. concrete pipe was inspected for corrosion. The inspection detected the loss of thickness (most severe was located between 1 and 2 o'clock in the pipe), as well as holes in the pipe wall and voids in the soil behind them, i.e. pipe was missing all together (Figure 96). The laser light could be seen 8 in. from the expected internal diameter mark. Deviation of the actual internal radius from the expected was depicted in color (Figure 97) using a user defined topographic scale (yellow-orange-red where the actual radius is larger, and shades of blue where it is smaller)

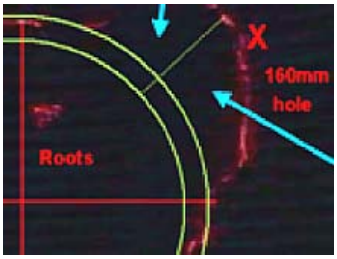


Figure 96. Hole in the pipe and void in the soil (Detail from Bennett and Logan, 2005)

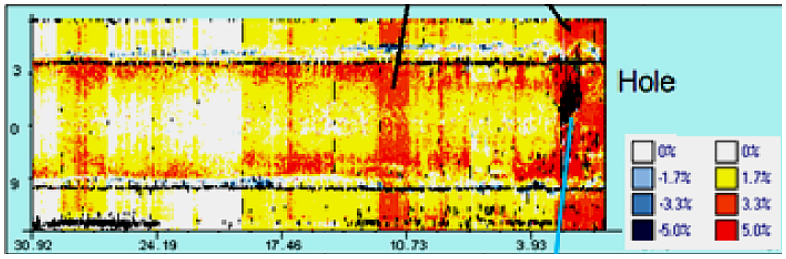


Figure 97. Corrosion in the concrete pipe is depicted in color that indicates the severity (Detail from Bennett and Logan, 2005)

City of Portland, OR (Bennett and Logan, 2005) used the method to evaluate distortion of CIP liners being installed in a 24 in. sewer pipe. Along the length of pipe, the "lift" of the CIP liner was measured (Figure 98) and was found to be up to 1.9 in. (8.4%) in some sections of the pipe. The profiling identified sections of failed CIP installation and eliminated the need to remove the entire installed CIP liner.

Lightman (2006) described a pilot study in Fort Worth, TX, which demonstrated great success of pipe profiling in showing corrosion and pipe deformations in large diameter sewers.

3-D laser profiling. 3-D lasers are a new generation of laser-based pipe inspection technologies (Redzone Robotics, 2008a) based on the principles of Light Detection and Ranging (LIDAR). A pulse of light emitted from the transmitter hits a surface such as a pipe wall, where part of it is reflected back to the receiver and detected with a built-in receiver. Because light moves at a constant rate through air, the distance to the reflecting surface is proportional to the time between the pulse emission and detection. Times can be measured very precisely, and thus so can the distances to pipe walls. 3D lasers create an image of an entire segment of pipe with a single scan (Figure 99).

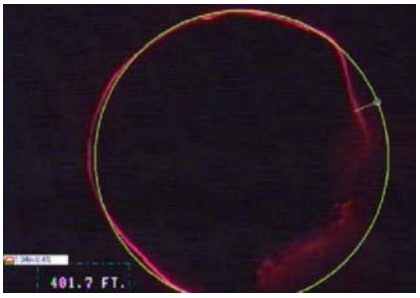


Figure 98. The lift of the CIP liner – an example of location with max lift (Detail from Bennett and Logan, 2005)

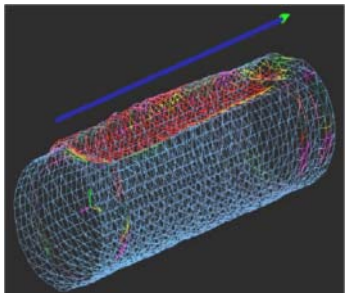


Figure 99. 3-D pipe plot (Dettmer, 2007)

2.4.2.4 Pigs

Pigs such as Go-NoGo gages can be used for contact type validation of pipe shape deformations over allowed tolerances. The gage, whose diameter is selected based on the allowed tolerance for pipe deformation, is pulled through a pipe and, if the pipe has larger deformation than the tolerance allowed, the gage gets stuck in the pipe. For instance, the allowed tolerance for plastic pipe is 5% (ASTM D 2412), and the gage's diameter would be 95% of pipe's nominal diameter. The testing does not return the severity of defect (i.e., the actual extent of shape deformation) or the number of such defects but rather identifies whether the pipe shape is acceptable or non-acceptable.

Figure 100. A 9-contact point mechanical wire framed Go-NoGo gage (Ouellet and Senecal, 2004)

2.4.3. STRESS-WAVE TECHNIQUES

Stress-wave techniques are based on the principle that a surface impact creates stress waves in the object being tested, which propagate through a material and, when an internal flaw or surface boundary is encountered, reflect back to the receiver. Various types of elastic waves (Figure 101) are produced after initial impact: compression waves (P-waves, longitudinal, particles motion parallel with the wave propagation) and shear waves (S-waves, transversal, particles motion perpendicular to the wave propagation) propagate into the object along spherical wavefronts, while surface waves travel along the surface away from the impact point (e.g., R-waves, Rayleigh, particles motion perpendicular to the surface and penetrating to a depth of one wavelength). The P- and S- stress waves are reflected by internal interfaces or external boundaries. Arrival of these reflected waves at the surface where the impact was generated produces displacements, which are measured by a receiving transducer. If the receiver is placed close to the impact point, the displacement waveform is dominated by the displacements caused by P-wave arrivals. The displacement waveform can be used to determine the travel time, t, from the initiation of the pulse to the arrival of the first P-wave reflection. If the P-wave speed, Cp, in the test medium is known, the distance, T, to the reflecting interface can be determined (Carino, 2008).

Existing stress-wave techniques differ in type of stress waves they measure and frequencies range. Only techniques that require access to one surface for placement of an impactor and a receiver are suitable for investigation of buried structures.

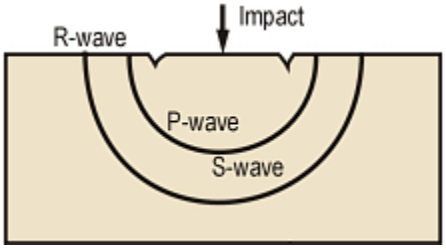


Figure 101. Stress waves due to an impact

2.4.3.1 Impact-echo (IE)

Impact-echo is a method used for flaw detection in concrete and masonry structures, and is applicable for concrete culverts. A short-duration mechanical impact such as tapping a small steel sphere against a concrete surface (Figure 104) is used to produce low-frequency stress waves (up to about 80 kHz) (IEI, 2005). The waves propagate into the structure and are reflected by flaws and/or external surfaces (Figure 102).

The wavelengths of these stress waves (IEI, 2005) are typically between 50mm and 2,000mm, which is longer than the scale of natural inhomogeneous regions in concrete (aggregate, air bubbles, micro-cracks, etc). As a result, the waves are only weakly attenuated, and they propagate through concrete matrix almost as though it was a homogeneous elastic medium. Multiple reflections of these waves within the structure excite local modes of vibration, and the resulting surface displacements are recorded by a transducer located adjacent to the location of impact (Figure 103). The piezoelectric crystal in the transducer produces a voltage proportional to displacement, and the resulting voltage-time signal (called a waveform) is digitized and transferred to a computer, where it is transformed mathematically (by Fast Fourier Transform) into a spectrum of amplitude vs. frequency. Both the waveform and spectrum are plotted on the computer screen. The dominant frequencies, which appear as peaks in the spectrum, are associated with multiple reflections of stress waves within the structure, or with flexural vibrations in thin or delaminated layers.

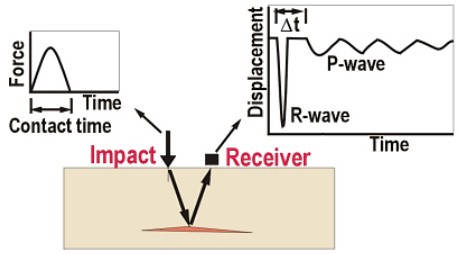


Figure 102. Principle of impact echo method (Redrawn from Carino, 2008)

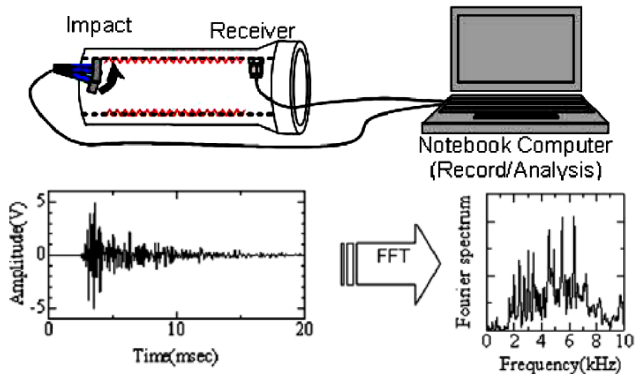


Figure 103. Outline of impact echo testing in a pipeline (Kamada and Okubo, 2005)

The fundamental equation of IE is $d=C/(2f)$, where d is the depth from which the stress waves are reflected (the depth of a flaw or the thickness of the pipe), C is the wave speed, and f is the dominant frequency of the signal. Details on the associated physical and mathematical principles can be found in a book by Sansalone and Streett (1997).

The duration of impact (called the contact time, shown in Figure 102) determines the size of defect that can be detected. As the contact time decreases, smaller defects can be detected, however, the penetrating ability of stress waves decreases. Commercially available impactors produce impacts with contact times typically 30-50 microseconds (IEI, 2005).

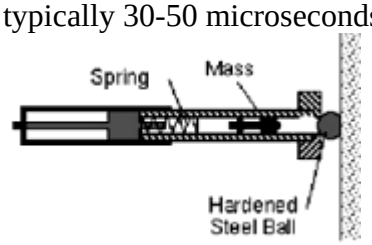


Figure 104. Spring-loaded mechanical impactor (Carino, 2008)

Figure 105. Impact echo applied manually in the field (IEI, 2005)

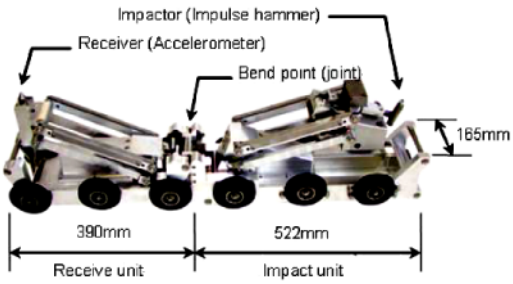


Figure 106. Inspection robot developed for impact echo testing (Kamada and Okubo, 2005)

Impact echo testing can be conducted in pipelines in two different ways: by using a hand-held single point device or by using a scanner system (Sack and Olson, 1994; Sack and Olson, 1998). Either way, the pipeline must be empty of water (i.e., isolated and dewatered, if necessary).

The <u>use of a hand-held single point device</u> requires man entry (Figure 105). Impacts need to be created at several locations around the circumference of the pipeline and at specified distances along the length

of the pipeline. It is a time-consuming process and the survey rates are limited (Higgins et al., 2003). Data can be collected at rates of 4-5 seconds per point or slower, depending on access conditions and the desired grid (Sack and Olson, 1998).

The scanning system described by Sack and Olson (1994) consists of four scanning heads spaced at 90° around the interior of the pipe circumference and pressed against the pipe wall pneumatically. The scanning heads have the rolling transducer assemblies (receivers) mounted adjacent to solenoid impactors. The scanner assembly is pulled through the pipe collecting data from each scanner head sequentially (liner spacing between tests is 2.5 in., testing rates in clean pipe about 4 ft/min).

Sack and Olson (1994) described results of the initial research into the use of IE scanning for testing of precast concrete cylinder pipe (PCCP). The method measured wall thickness of the pipe and compared to the design wall thickness. Losses between 0.75 in. and 1 in. were associated with the delamination of the outer mortar layer due to wire corrosion. It was shown that delamination of the grout layer caused by corroding wires is detectable with the IE method as long as the grout layer thickness is at least 4%-5% of the total wall thickness (the method resolution). The prototype scanner system used worked relatively well in a test section of buried damaged pipe, but had problems in field testing (in-situ pipeline) due to mud on the walls and lack of field hardening of the hardware.

Sack and Olson (1998) reviewed the performance of IE scanning and IE single point testing in the same PCCP pipe. The pipe ID was 72 in. and thickness 5.25 in.(the steel cylinder was embedded roughly 2.5 in. from the inside wall, while the outside grout layer was 0.75 in.-1.25 in.). The IE scanning obtained some usable data, but the unusually soft concrete (the surface could be scratched easily) created unacceptably high noise on the received signals that prevented analysis with the automatic scanning software and data had to be interpreted manually. The IE single point testing worked well even on softer concrete. One typical result was showing a single peak in the frequency domain spectrum at 12,800 Hz, corresponding to a thickness of 5.6 in. (a sound location in the pipe). Another typical result was showing two distinct frequency peaks (Figure 107): the lower frequency peak (18,000 Hz) corresponding to the thickness of 4 in. and indicating delamination of the outer grout, and the higher frequency peak (29,500 Hz) corresponding to the thickness of 2.5 in. and indicating debonding of “inside” concrete layer from the steel cylinder. Figure 108 shows the overall results from the single point testing: only three thickness echoes could generally be seen indicating that failures of PCCP pipes occur in predicable locations.

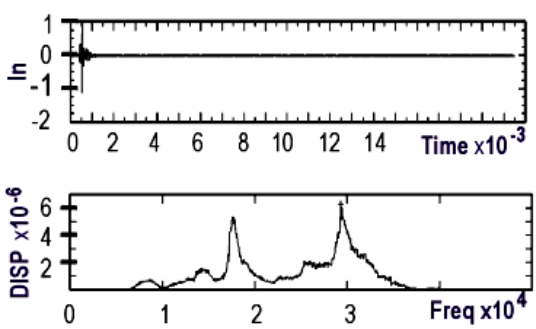


Figure 107. Typical record from delaminated conditions (Redrawn from Sack and Olson, 1998)

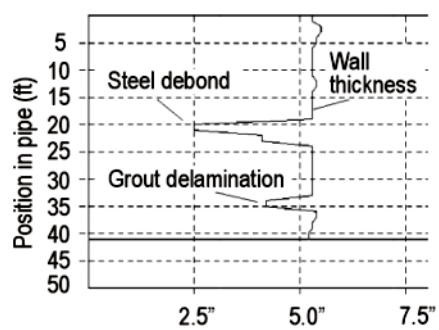


Figure 108. Overall results from the single point testing: wall thickness vs. location (Redrawn from Sack and Olson, 1998)

Kamada and Okubo (2005) described another robotic system for impact echo testing of pipelines developed in Japan. The diagnostic robot carries both an impactor (an impulse hammer) and a receiver (an accelerometer), and has a bend point between the impact unit and the receiving unit to allow insertion into the pipeline through tight spaces (Figure 106). This technology has been put to use in Japan but is not planned to enter the US market in the near future (personal communication with Manabu Okubo, Heitkamp, Inc)

2.4.3.2 Spectral Analysis of Surface Waves (SASW)

Spectral analysis of surface waves uses R-waves (Rayleigh, surface waves) to investigate conditions inside the concrete pipe and soil conditions outside of the pipe. The method is based on measuring propagation of R-waves in layered elastic media (Krstulovic-Opara et al., 1996; Sack and Olson, 1998).

SASW requires one surface to be accessible (Figure 109). For determination of thickness and stiffness of a concrete liner in the tunnel, the measurements are accurate within 5% (Force Technology, 2007).

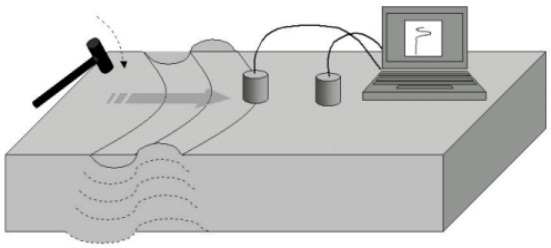


Figure 109. Principle of testing with SASW kit (Force Technology, 2007)

Figure 110. Testing with the SASW kit on the wall (Force Technology, 2007)

Sun et al. (2007) described one case study in Beijing, China, in which SASW was used to test the quality of road foundation above newly installed culvert (a concrete pipe approx 160 ft long, ID 60 in., 6.3 in. wall thickness, and installed with holes in the pipe crown through which the soil was grouted with a cement and fly ash slurry). The thickness of soil cover was about 23 ft. A hand hammer (0.5 kg, or approx 1 lb) was used to knock onto the pipe crown and create Rayleigh waves of desired frequencies. The SWG multiple seismic wave apparatus involved nine arrays of detectors, each array having 12 detectors spaced 20 ft apart, with end detectors overlapped for continuous data recording, fixed onto the pipe crown (Figure 111).

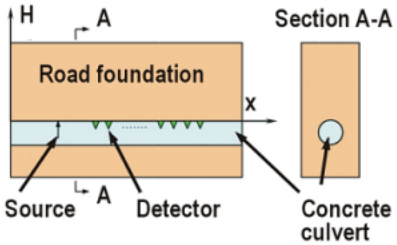


Figure 111. Schematic for observing a system of Rayleigh waves inside the concrete culvert (Sun et al., 2007)

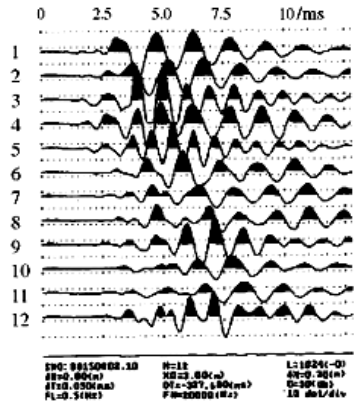


Figure 112. Waveforms recorded in one (the 8th) array (Sun et al., 2007)

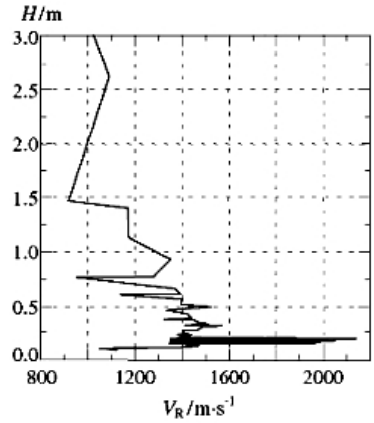


Figure 113. Phase velocity vs. depth curves, at x=127 ft (Sun et al., 2007)

Waveforms were recorded (9 arrays of waveforms, Figure 112) and phase velocity distribution along the depth between two adjacent measuring points was obtained (total of 99 curves of phase velocity vs. depth, Figure 113). The distribution of Rayleigh wave velocities in the road foundation across the length was formed by combining the curves of phase velocity vs. depth. Generally, the soil density is directly proportional to elastic wave velocity. The obtained results showed that (1) disturbance to the soil mass from culvert installation was very small, and (2) the soil near the culvert pipe was consolidated (from the grouting operation).

2.4.4. MODAL ANALYSIS TECHNIQUES (VIBRATIONAL TESTING)

Testing methods based on modal analysis analyze dynamic properties of structures under vibrational excitation. Two methods have been identified as applicable for culvert condition assessment.

2.4.4.1 Mechanical Impedance Testing

In mechanical impedance testing, a sinusoidal force is applied to the pipe surface to achieve a specified motion response (displacement, velocity, or acceleration).

A small shaker can be used to provide vibrations (Figure 114), while force and motion measuring transducers (separate or housed in a single instrument, an impedance head) measure simultaneously force input (lb) and motion output (in, in/sec, or in terms of g). Most often the measurements are made in terms of acceleration (Bradley, 1960).

The testing is performed by taking measurements in the center points of a grid. Measured input and output at any point in the pipe can be used to calculate the force to displacement ratio (lb/in) yielding the combined stiffness of the culvert and soil at that point. The stiffness values can indicate voids or overcompaction in the soil around the culvert. The grid resolution determines the minimum size of detectable voids (i.e., voids smaller than one half of distance between the grid points cannot be detected).

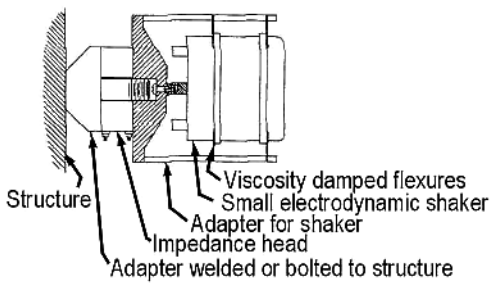


Figure 114. Impedance test arrangement (Redrawn from Bradley, 1960)

Figure 115. Impedance testing in corrugated steel culvert for MN DOT (CAN, 1996)

The vibratory force in the testing must be large enough to produce an accurately measurable motion but must not be so large as to damage the pipe. Most impedance tests are made in the frequency range 10 to 5,000 Hz (Bouche, 1961).

Example case study. MN DOT had the impedance testing carried out on a corrugated steel culvert in Nov 1994 and Aug 1995, prior to the culvert rehabilitation design (Sterling et al., 1997). A handheld shaker, 8 lb, was used to vibrate the culvert in a small area (Figure 115) applying a force of 20 lbs in most tests, and 40 lbs only in some tests. The grid points were spaced at distance 2 ft apart. Total of 115 measurements was performed in each testing (Figure 116). The sensors simultaneously measured force and acceleration. A Hewlett-Packard dynamic analyzer was used to collect the sensors output, integrate the acceleration to get velocity and displacement, and compute the force to displacement ratio, i.e. the stiffness.

Stiffness values (Figure 116) were found to be lower near the open end of the culvert showing that measurements taken at locations where there was no soil backfill produced the lowest stiffness. At 41 locations that were more than three grid segments from the culvert ends, the stiffness ranged from 47,000 lb/in to 236,000 lb/in, with an average of 147,000 lb/in and a standard deviation of 33,000 lb/in. Seven locations were more than one standard deviation below average, indicating possible soft soil or voids. Five locations had more than one standard deviation above average, indicating large rocks or overcompaction around the culvert. The soil during the Nov 1994 test was frozen yielding higher

stiffness values between 760,000 and 1,720,000 lbs/in.

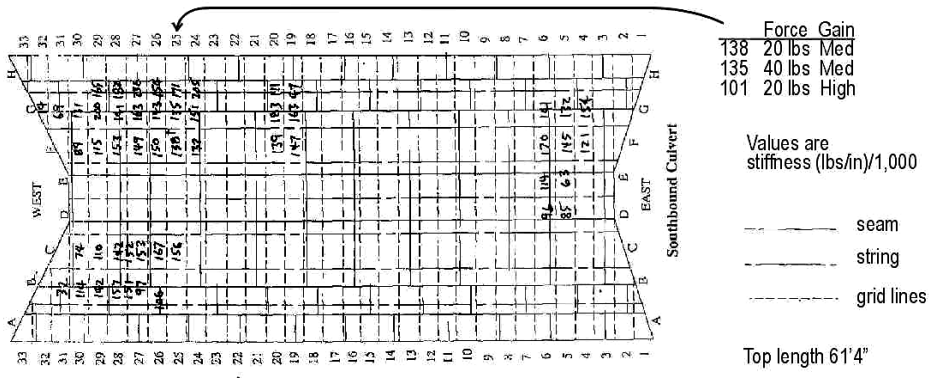


Figure 116. Stiffness values in grid points in the MN 1995 test (Redrawn from Sterling et al., 1997)

2.4.4.2 Natural Frequency Measurement

Natural frequency measurement is a method in which a pipe wall is vibrated in a range of frequencies while measuring the resulting pipe vibrations (Eldada, 1997; Makar, 1999). Frequencies that cause the largest vibrations are called the natural frequency.

In an undamaged pipe, certain characteristic natural frequencies are expected, while deviations from those frequencies indicate defects in the pipe wall or surrounding bedding. However, the frequencies are the global property of the pipe, and the method does not indicate the location of defects. The method is suitable to identify changes in the pipe condition over time and can show if there is an increase in damage.

It is essential to have an understanding of exactly how the natural frequencies of different types of pipe wall would be expected to change with increasing damage. However, other factors can also affect the results e.g., changes in bedding material or quality, the amount of water in the pipe and the height of ground water around the pipe. Considerable research is needed to determine if these effects can be separated from those produced by actual damage to the pipe wall (Makar, 1999).

2.4.4.3 Microdeflection Testing

Makar (1999) described briefly the principle of microdeflection testing method but did not provide details how the method is actually applied. These details are apparently contained in Eldada (1997), however, this report (in French only) was not obtained for this literature review.

Based on Makar (1999), microdeflection is the method in which pressure is applied to the inside surface of the pipe wall to very slightly deform it, and the deflection versus the increase in pressure applied is measured. In an undamaged concrete pipe, the material is expected to expand continuously (although not equally) in all directions as the pressure increases. However, a deflection increasing in one direction while decreasing in another, or a sudden change in the slope of a graph of applied force versus deflection, suggest that the pipe wall is damaged. The method can only be used in rigid pipes, i.e. concrete and VCP culverts, where an entire pipe wall is deflected by the applied force. It is not suitable for inspection of thermoplastic (PVC, HDPE) and metal culverts because local deformation of the pipe wall would tend to provide a false indication of the pipe condition.

One difficulty with this technique is determining the maximum safe pressure for use on a pipe wall so that the inspection method does not damage it. This pressure can readily be calculated for undamaged pipes but not for aged and deteriorated pipes. The safety consideration is not as important for concrete culvert pipes, where the strength of the pipe material is more uniform around the pipe circumference, then is for some other pipe materials (vitrified clay pipes, brick pipes, etc).

2.4.5. ULTRASONIC TESTING

Ultrasonic testing uses higher frequency stress waves compared to impact echo (20 kHz to 10 MHz and beyond, depending on the application). Most metals and plastics are inspected with frequencies typically between 2-10 MHz (occasionally up to 50 MHz).

Lower frequencies, typically between 25 and 150 kHz, are usually used for testing concrete. Higher frequency (above 100 kHz) can be used for small specimens, relatively short path lengths, or high-strength concrete, whereas lower frequency (below 25 kHz) are used for larger specimens and relatively longer paths, or concrete with larger size aggregates. The best frequency at which concrete can be characterized is around 150 kHz (Malhotra and Carino, 1991).

2.4.5.1 Drained Pipes

Ultrasonic testing can be carried out as contact and non-contact testing (Iyer and Sinha, 2005b). In contact testing, the transducer face makes a direct contact with the material being tested through a thin film of couplant. In non-contact testing, the waves are generated and transmitted into material to be tested without direct contact with the generating source. Only a portion of the sound is transmitted into the material (the proportion depends on how close the acoustic impedance of the two materials matches).

Regardless of the mode, ultrasound is reflected and transmitted at interfaces and through the test medium. Three different paths of ultrasound reflection and transmission can be distinguished (Bhardwaj, 2002):

- Single transducer operation - pulse echo (Figure 117)
- Separate transmitter and receiver operation on the same side - pitch-catch (Figure 118)
- Direct transmission (Figure 119)

It has been a long-standing challenge in the non-contact ultrasonic testing to produce high-resolution data using air-coupled ultrasound. However, in recent years, non-contact transducers have been successfully produced in the frequency range of 100 kHz to over 5 MHz. (Bhardwaj, 2002)

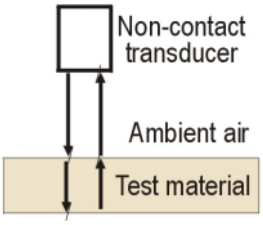


Figure 117. Pulse-echo testing (Redrawn from Bhardwaj, 2002)

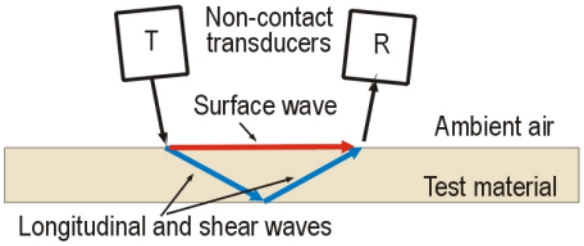


Figure 118. Pitch-catch testing (Redrawn from Bhardwaj, 2002)

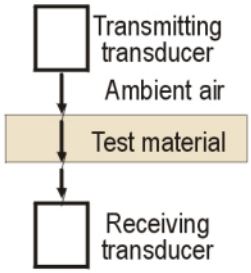


Figure 119. Direct transmission

Gaydecki et al. (1992) described the manner in which medium frequency (40-500 kHz) ultrasonic waves traveling through concrete are generated, received, digitized and analyzed. The scatter of ultrasonic waves due to non-homogeneity of concrete precludes the use of frequencies extending beyond 150 kHz, and frequencies below 100 kHz or 40 mm wavelength are generally employed in practice. Such low

frequencies cannot provide sufficient special resolution for conventional imaging. Thus, pulse-echo testing is generally considered unsuitable for testing of concrete and pitch-catch systems are predominantly used.

The use of ultrasonic technique for the non-contact inspection of buried pipes is still in early stages of preliminary research and experiments. Iyer and Sinha (2005b) proposed the use of this technique in combination with optical surface imaging to add complementary pipe information (a depth perception).

2.4.5.2 Undrained Pipes

Underwater ultrasonic inspection is very suitable for inspection of pipes with high flow conditions, and can further be enhanced by combining it with CCTV.

Pace (1994) described measurements which had been made to determine the acoustic properties of sewage and the characteristics of one sonar survey system undergoing trials. The underwater unit consisted of a steel tube, 20 in. long and 3.5 in. outside diameter on one end, which was mounted on a rotatable transducer. The entire unit was mounted on trolleys that could be winched through the pipe between access points, up to 1,000 ft apart. A frequency of 2 MHz was used for the central frequency of the sonar. Several units were reported being in use gaining operational experience.

Andrews (1998) described the use of high frequency (2 MHz) rotating sonar technology for scanning of a full wetted perimeter of sewer pipes in one case study in Canada, where over 30 miles of large diameter sewers (115 in. brick and 82 in.-95 in. concrete pipes) were inspected. Pulse-echo testing was used (i.e., an acoustic pulse was emitted from a transducer and the subsequent reflections of the pulse echo were received from some surface). The sonar transducer was mounted in an appropriate housing (Figure 120) and towed through the sewer. The acoustic signal, or pulse, of was transmitted radially toward the sewer wall using a rotating transducer and by analyzing the received echo, the distance from the transducer to the wall was calculated and the shape of the interior wetted perimeter determined. The on-board computer generated images of the interior perimeter in real time and produced a display.

The sonar equipment was deployed through the pipe like CCTV inspection equipment: the equipment was placed in a rig configured to the size of the sewer, a tow cable was used to advance the rig and sonar through the sewer, and an umbilical cable provided communication with the control and monitoring computer at the surface. In this case study, the sonar was used in conjunction with CCTV equipment and was suspended in the pipe below the rig (Figure 121)

Sonar and camera equipment proved capable of readily assessing existing sewer size as compared to as built dimensions, sediment build-up, and hydraulic roughness of pipes.

Figure 120. Ultrasonic inspection equipment (Andrews, 1998)

Figure 121. Floating rig with CCTV and lights. A sonar could be suspended below the pontoons under the camera (Andrews, 1998)

A research at King's College in London, UK, has been aimed at developing a novel approach for the internal inspection of sewers through the use of ultrasonic techniques (Gomez et al., 2003; 2004; 2006).

A simulation tool (simulation algorithms) capable of rendering images from pipes filled with water has been developed. The developed tool is capable of creating 2-D and 3-D surface scans of submerged pipe surface. Pipe deformations and anomalous conditions can be identified. The comparison of simulations against experimental tests validates the capability and accuracy of the generated ultrasound images.

Gomez et al. (2006) described an approach for the simulation of ultrasonic scans in water-filled pipes. The research provided insights into the behavior of the ultrasonic fields inside tubular structures.

2.4.6. GROUND PENETRATING RADAR (GPR)

2.4.6.1 Principle of Operation

Note: Surface Penetrating Radar is another term for this method that actually describes more accurately its application to various situations.

Ground Penetrating Radar (GPR) is a wave propagation technique that transmits electromagnetic (EM) waves through an antenna and collects signals reflected from a visually opaque substance or earth material. A typical GPR (Daniels, 1996a) generates a short impulse of electromagnetic energy (signals with central frequency typically between 50 MHz to 1.0 GHz), which is launched into the transmission medium (e.g., soil, concrete) via the transmission antenna. Energy reflected from interfaces between materials (discontinuities in impedance) is received by the receiving antenna, and is processed and displayed by a radar receiver and display unit (Figure 122).

The theory of radar wave propagation in solids (Bungley, 2004) is complex and described in text books, e.g. Daniels (1996b), Taylor (1994), etc; however, it can be simplified for most practical testing. The signal velocity is generally taken to be inversely proportional to the square root of the relative permeability (e.g., 1 for air, 81 for tap water, and between 5 and 12 for concrete). In practical situations, the relative permeability of materials is unknown and the propagation velocity must be measured in-situ or derived (back calculated) from multiple measurements. The depth to a target (or the thickness of the material) is calculated by multiplying the velocity of propagation by half of the round-trip transit time to and from the target (Daniels, 1996a).

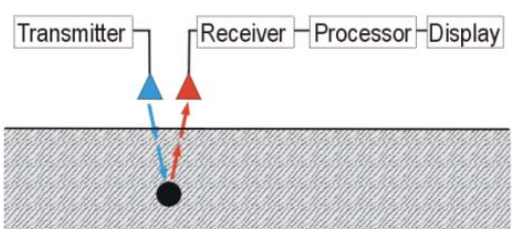


Figure 122. Block diagram of generic radar system (Redrawn from Daniels, 1996a)

Most GPR systems use separate antennas for transmission and reception (Daniels, 1996a). Additional constraint for impulse radar systems is the liner phase response, which restricts the class of suitable antennas to only several types (TEM horns, resistively loaded antennas, biconical antennas, loaded dipoles and bow-tie antennas).

GPR data can be collected and represented (Ozdemir et al., 2008) by using three different scanning geometries: A, B and C scans (1-D, 2-D and 3-D scan, respectively). In an A-scan, a single static measurement is performed by placing the antenna above a specific point. The A-scan data can be represented by a plot of signal strength versus time delay. In a B-scan, a series of A-scans is collected by moving the antenna(s) on top of the ground surface. 2-D B-scan raw data is usually acquired in frequency domain. GPR is most often used in the B-scan mode. In a C-scan, 3-D imaging is

accomplished by presenting data as three-dimensional blocks, or as horizontal or vertical slices. Horizontal slices (known as "depth slices" or "time slices") are essentially planview maps isolating specific depths (2-D images of the area are shown in top-down direction i.e., “birds eye view”).

GPR will detect, within the limits of the physics of propagation, changes in the electrical impedance of the material under investigation (Daniels, 1996a). The selection of antennas for GPR depends on the desired depth of penetration and feature resolution, and is typically a compromise between these two requirements. The maximum resolution achievable (Duran et al., 2002) is in the range of the value of the wavelength and maximum penetration depths typically do not exceed 20 wavelengths. For adequate penetration in moist environments, low frequencies need to be used.

One important advantage of GPR is the fact that the antenna does not have to be in contact with the surface of the pipe, and another is the fact that information about what surrounds the pipe and the interface of the pipe-soil interface can be obtained. Overall, leakage (i.e., wet soil) and metallic objects are easily detected. However, detection of cracks and air gaps in concrete or plastics can be difficult because the relative permitivity of these materials is very close to the permitivity of air (i.e., insufficient contrast). Another shortcoming of GPR systems in the ground is the “ground bounce” where significant amount of the energy is reflected from air-ground surface and thus does not propagate into the ground.

2.4.6.2 GPR Applied from the Ground Surface

Leak detection. Nakhkash and Mahmood-Zadeh (2004) demonstrated the feasibility of applying GPR from the ground surface to defect leaks from buried pipes. The objective of the study was to investigate effects of various factors, e.g., soil types, the presence of objects near leaking pipe and leakage size, on GPR signal. The study was performed as numerical simulation. The frequency of antennas used in this work was 200 MHz. The simulations were performed assuming a 6 in. concrete pipe buried in uniform dry sandy soil, at a depth of 47 in.

Nuzzo et al. (2008) addressed the problem of imaging leaking pipes by exploiting a novel microwave tomographic method, which takes advantage of the available knowledge of the investigated scenario (pipe position and size) and is thus able to detect the presence of leakage even in its early stages of development. For assessing the performance of the tomographic algorithm, a Finite-Difference Time-Domain (FDTD) forward modeling scheme was used to independently simulate realistic data collected during GPR survey over a leaking metallic water pipe. The basic geometry of the “pipe” model in this study involves a 120 mm (~5 in.) metallic pipe, filled with water and buried at depth 0.5 m (approximately 20 in.) to its center in a uniform dry sandy soil (Figure 123). Three phases of leak propagation have been modeled.

Voids detection. GPR systems can be used from the surface to determine the position of voids and anomalies below the surface, and measure the thickness of various layers in the soil. Provided several antennas are used and the velocity of propagation can be calibrated, it is possible to obtain accurate measurement of layer depth (Daniels, 1996a).

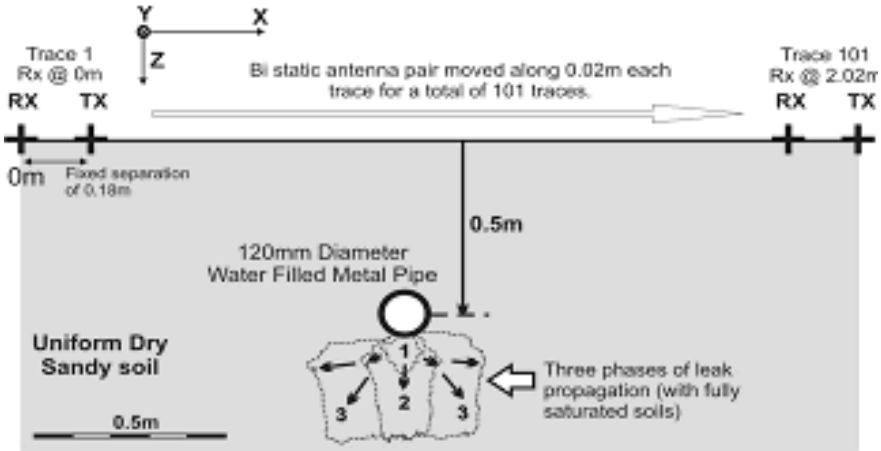


Figure 123. Basic geometry used in the study showing three phases of leak propagation (Nuzzo et al., 2008)

2.4.6.3 GPR Applied from Within Pipe

Voids detection in the soil envelope around the pipe. GPR systems can be used from inside the pipe to assess the condition of the material surrounding the pipe wall. The pipe should be constructed of non-metallic material. Daniels (1996a) described a test rig used to demonstrate the applicability of GPR for this purpose (Figure 124): an 8 in. pipe with a number of artificial voids was covered with soil, and a 6 in. pipe was positioned above the test pipe crossing its path. The radar image shows signal reflection from the 6 in. pipe, as well as 8 in. and 6 in. voids in the soil. The reflection from the interface between the soil and the air appears inverted in the radar image.

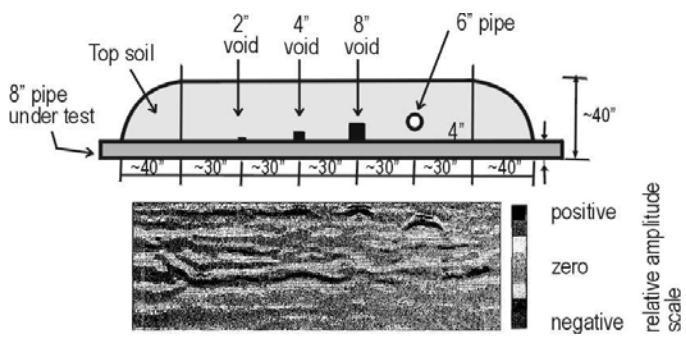


Figure 124. Test rig for GPR testing from within the pipe and radar image of defects adjacent to the pipe. i.e. voids in the soil (Redrawn from Daniels, 1996a)

Assessing the condition of (reinforced) concrete pipes. In one case study (Parkinson and Ekes, 2008), GPR was used to evaluate the condition of tunnel lining and locate concrete deterioration and voids in a continuous manner, by pulling the GPR unit through the pipeline. The tunnel (Kapoor Water Supply Tunnel, Victoria, Canada) was 8.8 km long (almost 30,000 ft) and circular (90 in. in diameter) except for the 200 m (656 ft) downstream section that had a horseshoe shape. The GPR employed a 1 GHz frequency antenna, mounted on a modular wooden pushcart (Figure 125), which was designed with a pair of angled 16 in. pneumatic wheels to run along the flanks of the tunnel. GPR identified a total of 401 anomalies in the concrete lining. Concrete honeycombing (this defect creates a distinct radar texture due to many scattering centers, i.e. patterns of many small-scale superimposed anomaly arches, Figure 126) was often detected in the surface of the tunnel crown area. GPR detected the concrete-rock interface (green line in Figure 127) and occasionally areas of liner-rock separation and occurrences of voids, which were showing as irregular “bright spots” with higher amplitude and lower frequency (red line in Figure 127). The amplitude of liner-rock reflection depended on the depth and curvature of the

interface (e.g., areas of thin concrete showed stronger reflections at the base compared with thicker concrete sections). One of the limitations is that GPR had to be placed very close to the pipe wall for a good resolution. It was also difficult to distinguish among various defects.

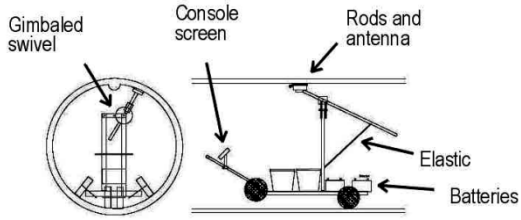


Figure 125. GPR on a push cart used from inside the tunnel (Redrawn from Parkinson and Ekes, 2008)

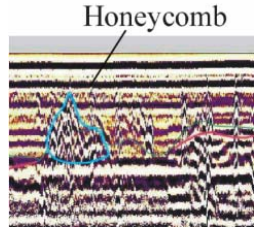


Figure 126. GPR image showing honeycombed concrete (Parkinson and Ekes, 2008)

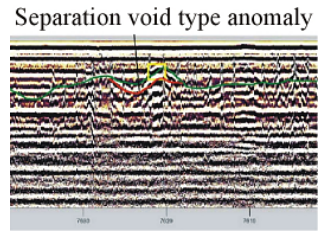


Figure 127. GPR image showing liner-rock separation anomaly (Parkinson and Ekes, 2008)

In another case study (Thiel et al., 2008), GPR was used in two tunnels in Tennessee, whose proposed rehabilitation required lowering of the tunnel profile elevation, to evaluate the thickness of the concrete liner (at the crown and spring line areas) and the depth below the pavement to the rock base. The tunnels were 30 ft wide. The GPR system used operated at 1 GHz frequency and employed a radar antenna with transmitter and receiver as separate components (Figure 128), and had a built-in distance measuring instrument. The system was suitable for both hand-held operation (for surveying of tunnel walls, Figure 129) and vehicle mounted surveying (for surveying of the crown and pavement). (Figure 130).

Figure 128. GPR antenna (Thiel et al., 2008)

Figure 129. GPR survey on the tunnel wall (Thiel et al., 2008)

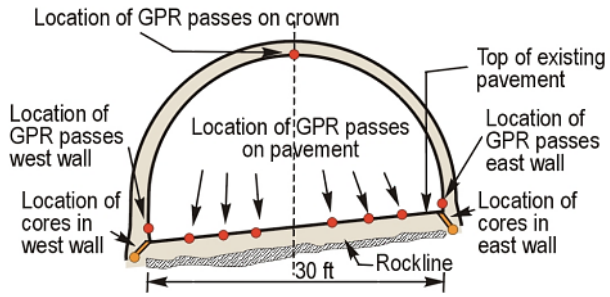


Figure 130. Location of GPR passes and cores (Redrawn from Thiel et al., 2008)

Koo and Ariaratnam (2006) reported the use of GPR in combination with optical scanning for the inspection of large diameter PVC lined concrete pipe in Phoenix, AZ. Two GPR units were attached to arms of the CCTV inspection robot (Figure 131), which enabled capturing data from any location above the pipe's spring line. Reflections of various amplitudes were produced at interfaces of materials with different electrical characteristics (Figure 132) indicating defects in the pipe material.

Figure 131. Two GPR units on a wheeled inspection robot (Koo and Ariaratnam, 2006)

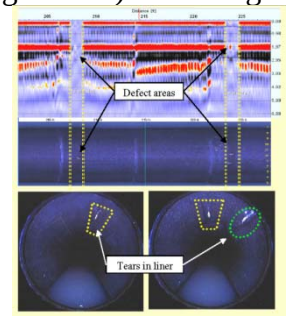


Figure 132. Results of GPR inspection from inside the pipe (top part of the image) and the location of defect areas (bottom pat of the image) (Koo and Ariaratnam, 2006)

2.4.7. ULTRA-WIDEBAND (UWB) RADARS

Jaganathan et al. (2006) described a novel inspection method that employs ultra-wideband (UWB) pulsed radar system for detecting "below surface" defects, corrosion and out-of-pipe voids in non-metallic buried pipes. The novel technology, developed at the Louisiana Tech University, is capable of transmitting and receiving electromagnetic pulses in the nano and picoseconds ranges. A numerical simulation study conducted on a VCP-soil interface showed that the technology is capable of precise measurement of pipe wall thickness in an automatic and continuous manner.

A consecutive study (Jaganathan et al., 2009) involved testing of the technology when applied within the VCP pipe for detecting the voids in the soil around the pipe. A numerical simulation of the propagation of UW pulses was performed using a three dimensional model (Figure 131) and laboratory testing in the soil bed was conducted (Figure 134) for validation purposes. Figure 135 shows an example of time-domain data produced by the UWB sensor, illustrating the relative simplicity of the data interpretation process.

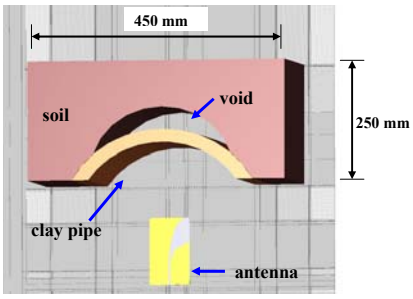


Figure 133. 3D numerical model of pipe inspection with soil void (Jaganathan et al., 2009)

Figure 134. Arrangement of antennas inside a 12 in. VCP pipe (Jaganathan et al., 2009)

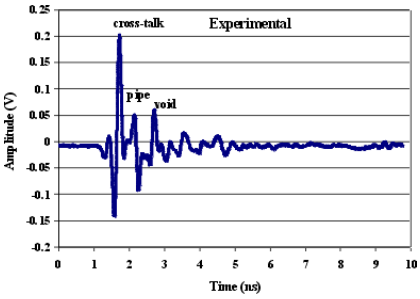


Figure 135. Experimentally measured back scattered signal for pipe with soil void (Jaganathan et al., 2009)

2.4.8. INFRARED (IR) THERMOGRAPHY

Infrared (IR) inspection method uses infrared cameras to "see" and "measure" thermal energy emitted from objects. Kouridakis (2007) provided a brief background (blackbody theory and infrared spectrum) and summarized the development of IR cameras. Thermal (or infrared) energy is the part of the electromagnetic spectrum that is not visible by the human eye and is perceived as heat (wavelength 20-0.7 microns, corresponding frequency range 0.15×10^{14}-4×10^{14} Hz). IR cameras detect IR energy (heat) of an area of the body (the field of view), convert it into an electronic signal and process to produce a thermal image on a video monitor and perform temperature calculations (Figure 136).

Figure 136. Infrared camera (http://www.flirthermography.com/cameras/camera/1124/)

Weil (1998) explained how the method can be applied for locate subsurface pipeline leaks, poor backfill, and voids surrounding pipelines caused by erosion. The method is based on the principle that energy flows from warmer to cooler areas (the heat transfer occurs through soils, backfill, pavement, etc primarily by conduction), and the fact that the resistance to the energy flow depends on the material (e.g., good backfill has the least resistance to energy conduction, poor backfill has the increased resistance, and air-filled voids prevent the convection currents completely). For method applicability, there must be a temperature differential in the ground. With adequate temperature differential, the

temperatures on the ground surface could serve as clear indicators of the subsurface conditions.

If the pipeline contains flow with fluid temperature above or below the ambient ground temperature, a temperature differential is provided. If the pipeline is empty, the method is still applicable but the time of day when the testing is scheduled becomes important. Due to ambient temperature changes between day and night (the surface is heating from the solar energy at daytime and cooling at night), usually the greatest thermal differential in the ground can be found in the midday or at night. If there are trees or objects on the ground creating shades, the testing should be done at night (personal communication with Gary Weil, EnTech Engineering Inc).

IR thermographic scanning was the way IR thermography was applied in the 1980s (due to the limitations of technology development at the time). Weil (1998) described the main components of the scanning system. At present time, however, digital sensors with high resolution are available (e.g., FLIR Systems' SC8000 infrared camera, 1024x1024) and IR camera snapshots have eliminated the need for scanning (Personal communication with Gary Weil, EnTech Engineering Inc). Kouridakis (2007) described recent technologies advances, which involve more sensitive cameras, more powerful software, and lower costs.

Case study #1. A drainage pipe under the airport pavement was investigated at one airport in New England in 1990 (Weil, 1998). The inspection was conducted late at night. It took three nights to complete the work on total area of 2,000,000 SF. Images were collected (Figure 137, Figure 138) Uncovered were 12 subsurface voids of varying sizes.

Figure 137. Visual photo of runway pavement (Weil, 1998)

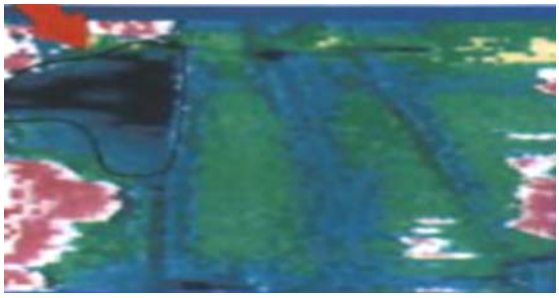


Figure 138. Thermogram showing the leaking drainage pipe (Weil, 1998)

Case study #2. A water pipe below the pavement was investigated in midtown St. Louis in 1983 (Weil, 1998). In addition to the water leak, an erosion area above the water pipeline was also found.

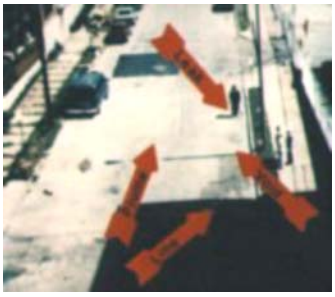


Figure 139. Visual photo (Weil, 1998)

Figure 140. Thermogram showing the water pipeline, water leak, water plum, and the void forming above the pipeline (Weil, 1998)

2.4.9. SUMMARY TABLE

Table 10 provides a summary of techniques for culvert inspection, outlining their applicability (culvert pipe type and flow) and ability to find defects.

Table 10. Methods for culvert inspection

Technique	Culvert type	Flow in pipe	The inspection will find:
Visual inspection of man-entry culverts	Any culvert type	No	Visible surface defects and defective joints. Also, pipe misalignment, shape or uniformity of curvature with additional field measurements.
Pigs	Any culvert type	Not important	Pipe shape deformations over allowed tolerances.
CCTV	Any culvert type	No	Visible surface cracks, deformation, defective joints, stains from corrosion, shape distortion
Optical scanning	Any culvert type, preferably not corrugated	No	Visible surface cracks, deformation, defective joints, stains from corrosion, shape distortion
Laser profiling	Any culvert type	No	Ovality, alignment, diameter. Also, defects such as surface cracks, corrosion of pipe inner surface, deposits
Impact-echo	Concrete culverts	No	Pipe wall thickness, delamination conditions within reinforced concrete pipe
SASW	Concrete culverts	No	Conditions inside the concrete pipe and soil conditions (density, voids) outside of the pipe.
Mechanical impedance	Any culvert type	No	Soil conditions outside of the pipe (voids or overcompaction in the soil around the culvert)
Natural frequency		No	Changes in overall pipe condition over time
Microdeflection	Concrete culverts	Yes	Damaged areas in pipe wall
Ultrasonic, pipes empty*	Any culvert type	No	
Ultrasonic, pipes full	Any culvert type	Yes	Pipe surface conditions and anomalities, deposits
Infrared	Any culvert type	Not important	Soil conditions outside of the pipe (voids, leakage from pipes)
GPR, from surface	Any culvert type	Not important	Soil conditions outside pipe (location, depth of voids)
GPR, from pipe	Any non-conductive culvert type	No	Defects behind liners

2.5. AGGREGATION AND UTILIZATION OF INSPECTION DATA

Data collected from the various inspection methods reported herein provide information regarding the presence, and at times, the severity of specific defects. The information collected from the various conventional and NDT inspection methods for a given culvert structure must be integrated using pre-established scales and rating methodologies to be able to compare the level of degradation of culvert structures with respect to each other as well as with respect to previous inspections of the same structure (indicative of deterioration rates). In addition, there is a need to convert the inspection data to recommendation for actions by the agency (i.e., replace, rehabilitate or do nothing).

* *Experimental stage only*

2.5.1. CULVERT Rating Systems

The following section provides an overview of culvert rating systems developed by transportation agencies across the US, which determine the remaining service life of culverts and the level of deterioration of culvert infrastructure.

FHWA's ***Culvert Inspection Manual*** (Arnoult, 1986) provides field inspection guidelines for identifying defects in culverts, rating the severity of defects, and assigning the condition ratings score.

FHWA's ***Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*** (FHWA, 1995) includes one item (item #62) that evaluates the alignment, settlement, joints, structural condition, scour, and other items associated with culverts. Inspection and rating of culverts is based on the FHWA's manual (Arnoult, 1986).

Kurdziel (1988) reviewed condition rating systems of metal culverts used in durability studies and publications in fourteen (14) different states. Once a metal culvert had deteriorated past superficial rust, there was little agreement on rating, and most studies did not show a uniform systematic progression of deterioration. For example, describing a condition as "moderate signs of deterioration" does not explain the condition, and specific degrees of deterioration should be listed (depth of rust, degree of pitting, etc). A new material rating system with detailed and unique description for each rating was proposed (Table 11). The degree of perforations spans over three ratings instead of one or two, which was the case in many state scales.

Table 11: Proposed metal condition ratings (Kurdziel, 1988).

Rating	Condition:	Description:
9	Excellent	New condition, galvanizing intact, no corrosion
8	Very good	Discoloration of surface, galvanizing partially gone
7	Good	Superficial or pinpoint rust spots, no pitting
6	Fair	Moderate rust, rust flakes tight, shallow pitting of surface, galvanizing gone
5	Fair-marginal	Heavy rust and scale, moderate pitting, slight thinning of core metal
4	Marginal	Extensive heavy rust, thick and scaling rust coatings, deep pitting, and significant metal loss (approx 25%)
3	Poor	Rust/pitting halfway through core metal (some deflection or penetration when struck with pick or geology hammer)
2	Very poor	Extreme deterioration and pitting, three quarters of core metal gone, first perforations
1	Critical	Extensive or large perforations
0	Failure	Invert completely deteriorated, culvert beginning to bend, warp or sag, collapse of culvert is imminent

The Pennsylvania DOT (PennDOT) performs an inventory and a condition survey of culverts along its highways statewide. Several parameters related to culverts are rated during the field inspection and entered into the Drainage eSTAMPP Survey Form including: physical condition, structural condition, flow condition, and roadway deflection (Table 12, Figure 141). PennDOT also rates appurtenances. Overall culvert condition is expressed in the form of single-digit number that describes the physical condition of the culvert (PennDOT, 2008).

Physical condition "2" Structural condition "3" Flow condition "2" Roadway deflection "1"

Figure 141: Some examples of PennDOT condition rating for different parameters (PennDOT, 2008).

Table 12: PennDOT criteria for condition rating of pipes (PennDOT, 2008).

Code	Physical condition	Structural condition	Flow condition	Roadway deflection
0	Like new, no defects	No displacement, as installed, good	Open	None
1	Mod to heavy rust/ pitting, large loss of inter protection, weathered joints.	Sag or structural components displacement <20° joint separation	< ½ clogged	> 1 in. surface distress
2	Broken, rust pitted, weathered joints	Sag or structural components displacement >20° joint separation	½ or more clogged	---
3	Extensive deterior., rotted to fully deteriorated bottom, loss of invert.	Collapsed or failed	Unknown (excess vegetation)	---
4	Unknown (excess vegetation)	Unknown (excess vegetation)	Unknown (inaccessibility)	---
5	Unknown (heavy flow)	Unknown (heavy flow)	"0" with PID	---
6	Unknown (complete clogging)	Unknown (complete clogging)	"1" with PID	---
7	Unknown (inaccessibility)	Unknown (inaccessibility)	"2" with PID	---
8	---	---	"3" with PID	PID= Possible Illicit Discharge)
9	---	---	"4" with PID	---

The California DOT (Caltrans) implements systematic condition evaluation only for culverts classified as bridges. Although there is no statewide culvert inspection program, Caltrans developed culvert inspection condition tables for the two-year culvert pilot project that was completed in June 2003. The rating system is in lieu of the FHWA rating system and is compatible with the Caltrans Culvert Inventory database. Condition rating of metal culvert barrels is rated on a 0 to 4 point scale based on waterway adequacy, shape, seams and joints, and culvert material. Feasible actions are listed for each condition rating (Table 13, Figure 142). (Caltrans, 2003)

Table 13. Caltrans criteria for condition rating of corrugated metal culvert barrels - steel and aluminum (Caltrans, 2003).

Condition	Waterway Adequacy	Shape	Seams, Joints	Material	Feasible Actions
0	No deficiencies	No deficiencies	No deficiencies	No deficiencies	0-Do Nothing
1	Minor debris and sediment, less than 25% blockage.	Good condition. Minor isolated distortions in top half. Minor flattening of invert. Horizontal diameter not greater than 10% of design.	Tight, no openings. Minor cracking at bolt holes.	- Steel: Superficial rust, minor pitting - Aluminum: Superficial corrosion, minor pitting	0-Do Nothing 1-Debris Removal 3-Flush Sediment 14-Invert Repair (Paving/Arm) 18-Undetermined 19-Other
2	Significant debris and sediment, between 25% and 50% blockage.	Significant distortion at isolated locations in top half. Significant flattening of invert. Some kinks present. Horizontal diameter greater than 15% of design.	Significant cracking at bolt holes. Partial separation at seems. Infiltration of backfill through seams and joints.	- Steel: Scattered heavy rusting and deep pitting. - Aluminum: Scattered heavy corrosion and deep pitting.	0-Do Nothing 1-Debris Removal 3-Flush Sediment 12-Joint Sealing 14-Invert Repair (Paving/Arm) 15-Culvert Barrel Lining 16-Replace a Section Barrel 18-Undetermined 19-Other
3	Between 50% and 75% blockage. Flooding of roadway and/or adjacent properties.	Major distortions throughout length of pipe. Major kinks and deflections. Flattening of crown and/or invert. Horizontal diameter greater than 20% of design.	Major cracking at bolt holes. Deflections of seams. Open joints. Infiltration of backfill.	- Steel: Extreme rusting, deep pitting, perforations present. - Aluminum: Extreme corrosion, deep pitting, perforations present.	1-Debris Removal 3-Flush Sediment 12-Joint Sealing 14-Invert Repair (Paving/Arm) 15-Culvert Barrel Lining 16-Replace a Section Barrel 17-Replace Culvert Barrel 18-Undetermined 19-Other
4	Over 75% blockage.	Partially collapsed or complete collapse of crown.	Failed	- S: Extensive rust, perforations. - A: Extensive corrosion, perforations	17-Replace Culvert Barrel 18-Undetermined 19-Other

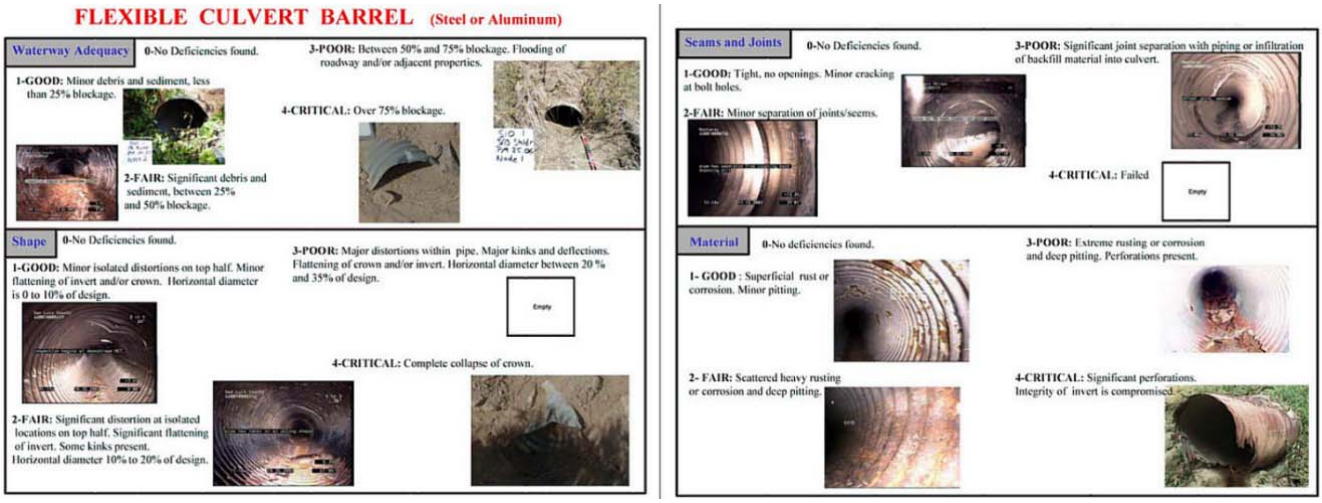


Figure 142: Caltrans criteria for condition rating of corrugated metal culvert barrel (Caltrans, 2003).

The Minnesota DOT (MN/DOT) performs inventory and condition survey of culverts utilizing the HydInfra management system. This asset management system (AMS) was launched in 1996 and has since been used for managing inventory, inspection and maintenance activities on state highway drainage systems. An inspector is asked to indicate a distress or a condition of the culvert by coding a number of parameters as Yes/No. The structural condition of pipe is rated on a 0 to 4 point scale (Table 14). Pipe Inventory Inspection and Maintenance Report is a typical output from HydInfra showing specifics of culverts (and other components of the system) and the result of their inspection (MN/DOT, 2006).

Table 14. MN/DOT's coding in HydInfra for culvert inspection (MN/DOT, 2006)

Code	Parameter	Code	Parameter	
Y or N	Plugged?	Y or N	Misalignment?	Codes for structural condition: 0 = Not ratable 1 = Like new 2 = Structurally sound 3 = Extensive deterioration 4 = Critical deterioration
Y or N	Deformed?	Y or N	Joints Separation?	
Y or N	Standing Water?	Y or N	Holes?	
Y or N	Infiltration?	Y or N	Inslope Cavity?	
Y or N	Silt Present?	Y or N	Road Void?	
Y or N	Piping?	Y or N	Road Stress?	
Y or N	Cracks?	Y or N	Erosion?	
Y or N	Spalling / Flaking?	0-1-2-3-4	Structural condition?	
Y or N	Pitting / Rusting?			

The Ohio Department of Transportation (ODOT) has a culvert inspection rating system that rates 16 items on a 0 to 9 point scale (Table 15). The condition of the ends of the culvert does not govern the condition rating. The inspector selects the lowest rating that best describes either the shape condition or the barrel condition (ODOT, 2003).

Table 15. Corrugated metal culvert coding for the culvert inspection report form (ODOT, 2003).

Code	Category	Description
9	Excellent	New condition; galvanizing intact; no corrosion.
8	Very Good	Discoloration of surface; galvanizing partially gone along invert but no layers of rust.
7	Good	Discoloration of surface, galvanizing gone along invert but no layers of rust. Minor pinholes (with an area less than 3 in^2/ft^2) in pipe material located at ends of pipe (length ≤4 ft and not located beneath roadway).
6	Satisfactory	Galvanizing gone along invert with layers of rust. Sporadic pitting of invert. Minor pinholes (with an area less than 6 in^2/ft^2, 4%) in pipe material located at ends of pipe (length not to exceed 4 ft and not located beneath roadway).
5	Fair	Heavy rust and scale. Pinholes (with an area less than 15 in^2/ft^2, 10%) throughout pipe material. Section loss and perforations, holes in metal located at ends of pipe in invert (not located under roadway).
4	Poor	Extensive heavy rust; thick and scaling rust throughout pipe; deep pitting; perforations throughout invert with an area less than 30 in^2/ft^2, 20%. Overall thin metal, allows for an easy puncture with chipping hammer.
3	Serious	Extensive heavy rust; thick and scaling rust throughout pipe; deep pitting. Perforations throughout invert with an area < 36 in^2/ft^2, 25%. Overall thin metal, easy puncture with hammer. End section corroded away
2	Critical	Perforations throughout invert with an area greater than 36 in^2/ft^2, 25%.
1	Imminent Failure	Pipe partially collapsed.
0	Failed	Total failure of pipe.

Cahoon et al. (2002) investigated 33 parameters that describe the condition of an existing culvert for possible use as predictors of overall culvert condition. These 33 parameters were recorded for 460 culverts distributed geographically throughout Montana. An ordered probit statistical model indicated that 9 of the initial 33 potential predictors were statistically significant (Table 16). Measurements of these nine parameters can be used in the resulting model to classify a culvert into a 1-to-5 condition ranking, in which 5 is the best condition and 1 is the worst. The model matched the overall rating assigned by the field observers in 61.7% of culverts observed.

Table 16. Statistical modeling of culvert condition rating (Cahoon et al., 2002)

Predictors statistically significant	Input values
1. Age of culvert	Integer years
2. Scout at outlet	0, 1 or 2
3. Evidence of major failure	0 or 1
4. Degree of corrosion	0, 1 or 2
5. Invert of culvert worn away	0, 1 or 2
6. Sedimentation of cross-section	0 to 100%
7. Physical blockage	0 to 100%
8. Joint separation	0 or 1
9. Physical damage	0, 1 or 2

The Ohio Research Institute for Transportation and the Environment (ORITE) proposed a new culvert inspection rating system that considers 30-33 items on the 0 to 9 point scale. The system was developed from data collected at sixty culvert sites, of which 25 were metal culverts. The statistical analysis indicated that age, rise, flow abrasiveness, pH, flow velocity and culvert material type were significant variables for the culvert rating system of (Mitchell et al., 2005).

2.5.2. CULVERT INSPECTION POLICIES (FREQUENCY OF INSPECTIONS)

Ring (1984) had suggested that culverts should be inspected at least every three (3) years, and more often where field conditions are harsh.

The FHWA's ***Culvert Inspection Manual*** (Arnoult, 1986) recommends that all culverts are inventoried and periodically inspected. Culverts longer than 20 ft must be inspected once every two (2) years,

however, the states may perform the inspections less frequently with the FHWA's approval, which is issued on a case-by-case basis if justified. For instance, if conditions are mild, the FHWA may approve inspection every four (4) years. Smaller culverts (less than 20 ft long) may not warrant the same rigorous level of inspection as large culverts, however, their failure can be life-threatening and they should still preferably be regularly inspected as well.

NCHRP Synthesis 303 (Wyant, 2002) indicates that as of 2002, there was no standard state and local culvert inspection cycle being followed by all highway agencies. There were more state DOTs with guidelines (37%) than local agencies (33%) and federal agencies (25%). Most local agencies responding to the survey indicated that they use the guidelines outlined in FHWA's Culvert Inspection Manual (1986). These conclusions are based on results from national survey on culvert inspection policies and procedures to which a total of 75 agencies replied.

Texas DOT (TxDOT) requires routine inspection of culverts every four years (TxDOT, 2002).

In Ohio, all bridges with spans greater than 10 ft are required to be inspected annually, however no Federal or State requirements mandate frequent inspections of short span culverts. Culverts less than 10 ft in span are inspected sporadically under varying Ohio Department of Transportation (ODOT) district procedures. The ODOT recommends that highway culverts having span between 1 and 10 ft are inspected once every five years (ODOT, 2003).

Another nationwide survey conducted in 2003-2004 by the *Ohio Research Institute for Transportation and Environment (ORITE)* indicates that approx 60% of state DOTs have developed culvert inspection policies. The majority of these state DOTs specified 1-2 year inspection cycle and a small percentage specified a 3-5 year cycle. Some states have dual frequency requirements (e.g., MN/DOT inspects large culverts with span >10 ft in a 1-2 year cycle and smaller culverts in a 5-year cycle; VADOT inspects large culverts (span >10 ft) in a 2-year cycle and smaller culverts in a 4-year cycle). Most of the states that inspect culverts apply a numerical rating system (Mitchell et al., 2005).

The inspection program used by the *New York State DOT* has a scale from 1 to 7, along with 8 for not applicable cases and 9 for conditions unknown cases. Culverts rated 5, 6 or 7 are inspected every forth year, those rated 3 or 4 are inspected every second year, while those rated 1 or 2 are rated annually. (NYSDOT, 2008).

Meegoda et al. (2004) developed inspection frequency guidelines that rate CSP culverts at three levels based on several factors (Table 17). A four-point condition state assessment system was developed (based upon the CalTrans system), that includes quantifiable section losses, specific surface features, and prescribed responses associated with each condition state (Table 18). A Markovian deterioration model was used to predict the future condition state of new CSP culverts in urban and rural settings. The transition probabilities were based upon inspection data and corrosion studies. The model was extended to predict the future condition of new CSP culverts in both settings over a 30-year life. The model does not take into account the effects of maintenance or rehabilitation.

Table 17. Proposed inspection frequency for CSP culverts (Meegoda et al., 2004)

Rating Level	I	II	III
Inspection Frequency	10 yrs	3 yrs	1 yr
Basis for time interval	Self-cleaning design (10-year flood) for small diameter CSPs	FHWA Guidelines	Reported problems
Corrosion/sediment	Free of corrosion and debris	Evidence of corrosion and/or debris	Reported clogging or collapse
Abrasion	Low abrasion- Minor bedloads of sand and gravel V < 1.5 m/s	Moderate abrasion. Bedloads of sand, gravel 1.5 m/s < V < 5 m/s	Severe abrasion. Heavy bedloads of gravel, rock V > 5 m/s
pH:	5.8 < pH < 8.0	5.0 < pH < 5.8	pH < 5.0
Corrosion/Erosion (Conductivity	Low or none	Medium	High

Table 17. Proposed inspection frequency for CSP culverts (Meegoda et al., 2004)

Rating Level	I	II	III
maps, historical data)			
Pipe Age	10 yrs	15 yrs	30 yrs (Design Life)

Table 18. Condition state assessment system (Meegoda et al., 2004)

Condition state	Suggested corrective action:
1. No evidence of active corrosion of the structure with any measurable section loss.	Do nothing
2. Surface or freckled rust is formed on the structure, flaking, minor section loss ≤10% of thickness	Clean and paint
3. Flaking and swelling with surface pitting but any section loss due to active corrosion is measurable and does not affect the strength or serviceability of the structure. Section loss: 10 - 30 % of thickness.	Clean and paint or re-lining.
4. Corrosion is advanced, heavy section loss >30% of section thickness.	Reline or replace

Meegoda et al. (2005) proposed a new culvert information management system (CIMS), which would be a subsystem of New Jersey DOT's transportation asset management system (TAMS). The proposed CIMS is based on the condition state of the culvert during previous year and the predicted survival probability of the CSP with service time data developed from an ASTM study (the study of corrosion of carbon steel from 1960 to 1964 at 46 locations, including 14 locations in other countries, as referenced in Meegoda et al., 2004 and described in Beaton and Stratful, 1962). The proposed CIMS can analyze decisions to inspect, rehabilitate or replace culverts, or do nothing, at both project and network levels. At the project level, inspection or rehabilitation and replacement costs are compared with failure risks and costs. At the network level, the costs are optimized to meet the annual maintenance budget by prioritizing needed inspection, rehabilitation and replacement activities.

Bhattachar et al. (2007) developed a framework for culvert inventory and inspection by providing necessary protocols and condition rating systems. Culvert inventory data collection consists of 55 questions grouped in six (6) modules. The basic condition assessment also has six (6) components (Table 19). Any culvert with performance score below 2.5 (red zone) requires an advanced condition assessment i.e., it is inspected for specific problems that have caused the deterioration.

Table 19. Modules for culvert inventory and inspection (Bhattachar et al., 2007)

Culvert inventory modules	Basic condition assessment modules
1. General identification of the culvert location	1. General identification of the culvert location
2. Structural information	2. Site information (climate, water level, ph, soil resistivity, etc)
3. Additional information to identify culvert components	3. Culvert identification (shape, material, end treatment, etc)
4. Hydraulics of the culvert	4. Condition assessment (condition of inverts, end protection, roadway, embankment, footings, overall culvert condition)
5. Safety features	5. Performance score, calculated using relative weights for all components
6. Culvert inventory (identification of past repair/rehab)	6. Zoning of culverts based on performance score: >3.5 indicates green zone (safe); between 2.5-3.5 yellow zone (intermediate); <2.5 red zone (danger).

2.5.3. DURABILITY AND Service Life

NCHRP's *Synthesis of Highway Practice 50*: *Durability of Drainage Pipe* (NCHRP, 1978) defines durability as the material's ability to resist degradation as a result of chemical or electrochemical corrosion and mechanical abrasion. When limited by material performance, useful service life of culverts depends on their durability. Culvert durability is usually affected by two mechanisms –

corrosion and abrasion. The synthesis stated that culvert durability is usually affected by two mechanisms – corrosion and abrasion (the review failed to mention external erosion of the backfill though it is arguably the most critical deterioration mechanism). The manual also pointed out that there is no widely agreed upon definition for failure of culvert, short of collapse. One way of defining the service life of a culvert is by the number of years of relatively maintenance-free performance. The culvert that has reached its service life may still have many years until failure.

Bealey (1984) reported that durability of culverts was identified as an issue of concern by more than 60% of state transportation agencies. A total of 33 states and numerous researchers had published 131 reports on durability of pipe materials, of which 63% were concerned with corrugated metal pipe.

FHWA's *Durability of Special Coatings for Corrugated Steel Pipe.* Potter, Lewandowski and White (1991) investigated if various coatings applied to plain galvanized CSP can give culverts the desired design life of at least 50 years. The study showed that which were bituminous coated and paved culverts, polymer coated culverts (ethylene acrylic acid film) or concrete lined culverts, under proper conditions, can have their expected service life extended to at least 50 years.

NCHRP's *Synthesis of Highway Practice 254: Service Life of Drainage Pipe.* Gabriel and Moran (1998) provided details on elements influencing material durability considered in the selection of drainage pipe. These elements include life expectancy of various types of pipe protection systems in different environments based on parameters such as pH, resistivity, abrasion, and flow conditions. Protection strategies that influence material durability were also addressed. The report cited the usage of thermosetting liners (cured-in-place liners) and pre-formed thermoplastic materials (HDPE or PVC liners) as a significant change, and noted that linings for metal culverts continued to encounter durability problems.

FHWA's *Durability Analysis of Aluminized Type 2 Corrugated Metal Pipe*. Ault and Ellor (2000) reviewed various methods used for predicting culvert durability. Two of the most commonly used methods for predicting the durability of metal culverts in soils are the California Test Method 643 and the ANSI/AWWA method.These methods estimate the combined effects of soil corrosion, water corrosion and abrasion on durability.

California Test Method 643 (Caltrans, 1999) calculates combined effects of soil corrosion, water corrosion and abrasion on durability of galvanized CSP culverts that have not yet received maintenance treatment. The method was originally developed in 1959 based on corrosion testing of 7,000 corrugated metal culverts located in one area of California (Beaton and Stratfull, 1962), but was refined over the years having the last update in 1999. Two environmental factors are combined for estimating the service life (years to perforation) of steel culverts: (1) pH of soil and water; and, (2) the minimum electrical resistivity of the site and backfill materials (Table 20). Using these parameters, the probable maintenance-free service life of a galvanized steel culvert in a given location can be estimated by using a chart (Figure 143). The service life is characterized as years to first perforation for an 18-gage galvanized CSP (2-oz per sq.ft. zinc coating). A correction factor is given for different culvert pipe thickness (gage between 18 and 8).

Table 20: Typical resistivity values (Wilson and Oates, 1969)

SOIL Classification	ohm-cm	WATER Source	ohm-cm
Clay	750 - 2,000	Seawater	25
Loam	3,000 - 10,000	Brackish	2,000
Gravel	10,000 - 30,000	Drinking water	4,000 +
Sand	30,000 - 50,000	Surface water	5,000 +
Rock	50,000 - infinity*	Distilled water	infinity*

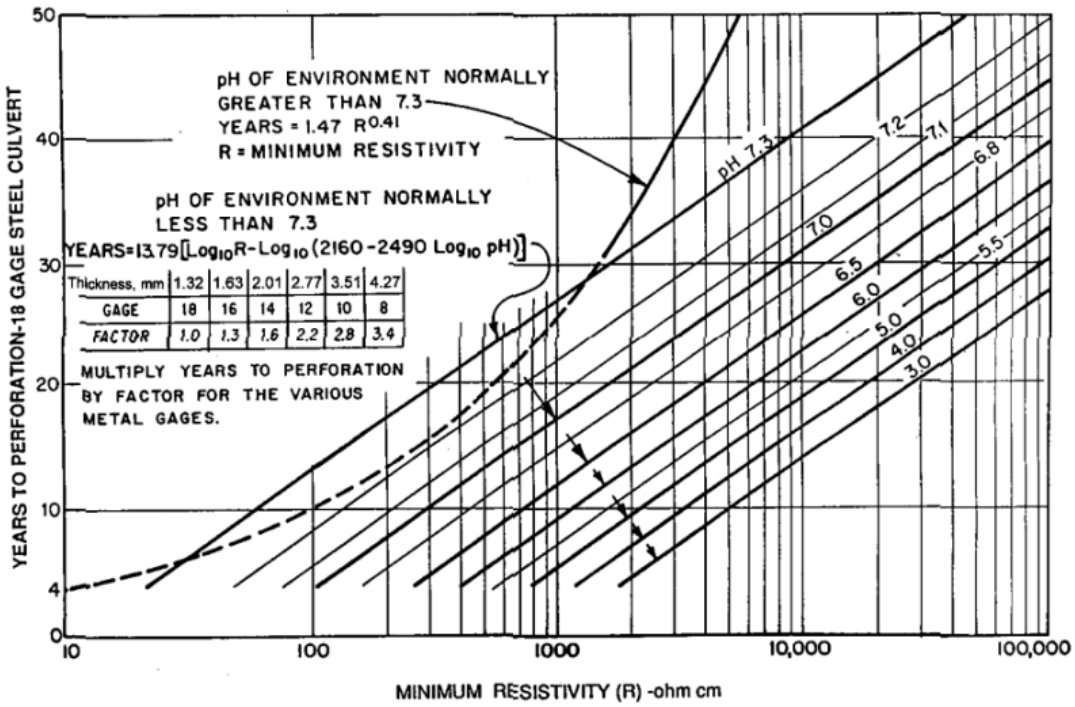


Figure 143. California Method chart for estimating years to perforation (Caltrans, 1999)

American Iron and Steel Institute (AISI) Method (AISI, 1994; CSPI, 2007) is similar to the California Method except that the vertical axis is expressed as the invert's average years of service life. It is believed that the consequences of small perforations in storm sewers are usually minimal. AISI Method takes the position that service life is limited by 25% average metal loss in the invert, whereas only 13% average metal loss occurs at first perforation. Therefore culvert durability predicted with the AISI Method is about double than obtained using the California Method (Ault and Ellor, 2000).

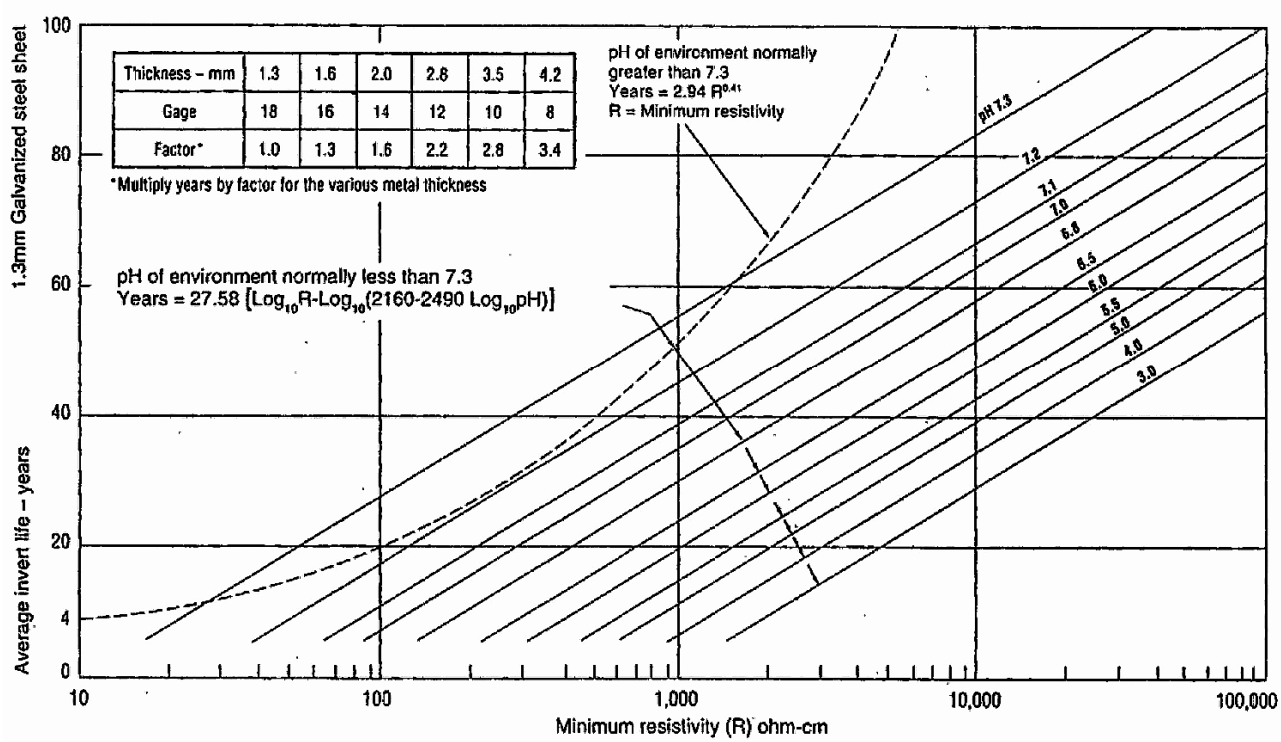


Figure 144. AISI Method chart for estimating years of service life (CSPI, 2007)

Florida Method was derived from the California Method and predicts durability of Aluminized Type-2 coated corrugated steel, concrete, and aluminum alloy culverts. A service life estimation graph is shown in Figure 145. Florida DOT has established that the life of Aluminized Type-2 coating is 2.9 times that of galvanized coating(Ault and Ellor, 2000 referencing Cerlanek and Powers, 1993)

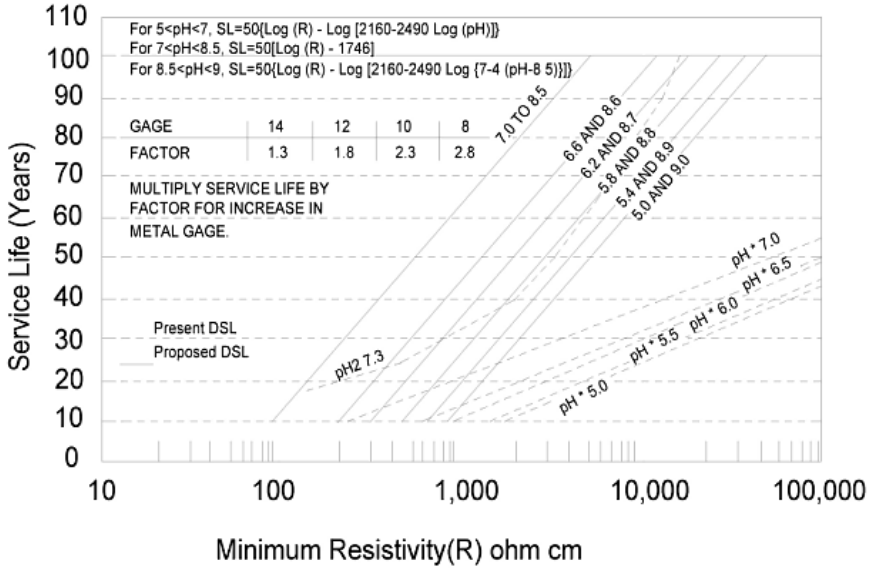


Figure 145. Florida method for estimating years of service life of Aluminized Type-2 coated pipes. (Ault and Ellor, 2000 referencing Cerlanek and Powers, 1993)

Florida DOT has developed a software program (Figure 146) for determining types of culvert material, whose expected service life (ESL) will meet or exceed the required design service life (DSL). The DSL, pipe size, pH, resistivity, chlorides, and sulfates are input variables, and the program provides a listing of those materials that meet the required DSL.

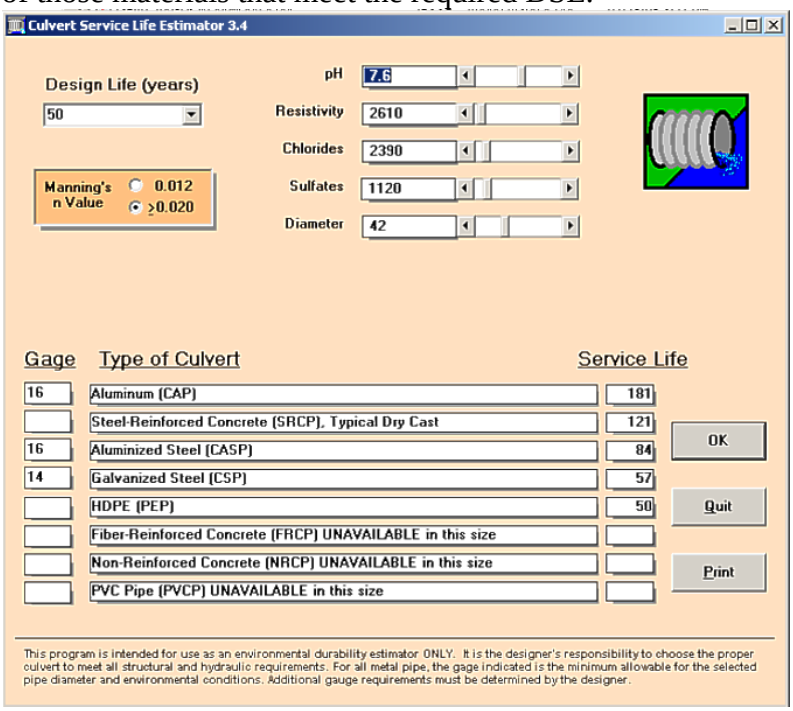


Figure 146. Culvert Service Life Estimator (FDOT, 2008a)

NCSPA's ***CSP Durability Guide*** (NCSPA, 2000) includes AISI chart for predicting service life of CSP and provides a table showing the added service life for various coatings at four abrasion levels. A total of 10 non-metallic coatings are included (Table 21).

Table 21. Estimated added service life for non-metallic coatings at different abrasion levels in years (NCSPA, 2000)

Abrasion levels	Level 1 & 2	Level 3	Level 4
Asphalt Coated	10	N/R	N/R
Asphalt Coated and Paved	30	30	30
Polymerized Asphalt Invert Coated*	45	35	N/R
Polymer Precoat	80+	70	N/R
Polymer Precoat and Paved	80+	80+	30
Polymer Precoat with Polymerized Asphalt Invert Coated	80+	80+	30
Aramid Fiber Asphalt Coated	40	N/R	N/R
Aramid Fiber Asphalt Paved	50	40	N/R
High Strength Concrete Lined	75	50	N/R
Concrete Invert Paved (75mm (3 in.) cover)	80+	80+	50

(N/R = not recommended)

Guidelines to evaluate abrasion levels

1. Non-abrasive (no bedload regardless of velocity or storm sewer application)
2. Low abrasion (minor bedloads of sand and gravel and velocities of 5 ft/sec or less)
3. Moderate abrasion (bedloads of sand and small stone or gravel with velocities 5-15 ft/sec
4. Severe abrasion (heavy bedloads of gravel and rock with velocities > 15 ft/sec.

NCSPA's paper "New Approaches to Determining CSP Service Life" (NCSPA, 2006) recognized that environmental conditions have different effects on sacrificial coating (galvanized) and barrier coatings (aluminized and polymer). Galvanized coating is soluble in water. Soft water is more detrimental to galvanized coating than hard water because the later has an excess of $CaCO_3$, which is deposited on the pipe wall in the form of scale, thus protecting the underlying galvanized coating. NCSPA recommends galvanized pipe only in locations where $CaCO_3$ concentration is greater than 50ppm (which corresponds to the resistivity level ≥ 10,000 ohm-cm). When installed in the right environment conditions, galvanized CSP can provide a 50-year service life. Barrier coatings provide a more uniform and predictable service life across several predetermined ranges of pH and resistivity criteria. An aluminized coating can perform well for up to 75 years and a polymer coating can function for a minimum of 100 years.

2.5.4. CORROSION RESISTANCE

Missouri DOT (MoDOT) performed field inspections on 3,897 culverts throughout the state of Missouri, of which 2,255 were corrugated metal pipe (CMP). The study identified that 45.6% of the CMP pipes needed replacement. Some of the CMP deterioration could be attributed to a change in the pipe gage (Gift and Smith, 2000).

Kansas DOT (KS DOT) documented that more rapid deterioration of CMP has been occurring since the late 1970s when standards changed to allow lighter gage metal in pipe construction. While the lighter gage pipes may have had adequate structural support from surrounding soils, the change in standard was reported to lessen pipe design life by nearly 20 years because of less metal to corrode at the same corrosion rate. The data in the report supported the decision to prohibit the use of CMP for crossroad installations in some districts in Kansas (Stratton et al., 1990).

NCSPA had the corrosion resistance of polymer coated CSP investigated by the Corrpro Companies (NCSPA, 2002b). In Wisconsin, five polymer coated CSP culverts were inspected, as well as three other culverts (epoxy coated, aluminized steel pipe Type 2, and aluminum). All culverts were in severe corrosive environment (low pH, low electrical resistivity of soil and water, anaerobic sulfate reducing bacteria in organic rich soil). The polymer coated CSP performed equally well or better than the other materials/coating systems. Only the epoxy coated pipe showed signs of corrosion at one end.

2.5.5. ABRASION RESISTANCE

State of California Division of Highways evaluated the thickness of aluminum culverts required to achieve a 25-years maintenance free service life in abrasive flow conditions. For a 10-year storm and in flow conditions less than 7 fpm, both uncoated and bituminous-coated corrugated aluminum pipe can be used. Cross-drains are the exception: corrugated aluminum pipe must be bituminous coated or paved, and may be used only in flow conditions less than 5 fpm. Wear rates of corrugated aluminum pipes and steel pipes were compared: at the same thickness, aluminum pipe would perforate by abrasion 10 times sooner than steel pipe (Nordlin and Stratfull, 1965).

Caltrans ***Evaluation of Abrasion Resistance of Pipe and Pipe Lining Materials***. DeCou and Davies (2007) evaluated 18 different pipe materials for their resistance to abrasion over a 5-year period in a natural stream setting. Among the materials tested were galvanized CSP, Corrugated Aluminized Steel Pipe Type 2 (CASP), and several galvanized CSP pipes with different coatings. Abrasion wear rates of pipes, liners and linings in the field were found to be non-linear with time, but rather event driven and dependent on the number and size of events during any given year. None of the protective coatings for steel was found suitable in extremely abrasive environments with high flow velocities, but these results would have limited applicability to other sites statewide.

NCSPA had the abrasion resistance of polymer coated CSP investigated (NCSPA, 2002a). In New York, the field performance of 20 corrugated steel culverts was evaluated. All culverts featured asphalt paved over a polymer coating (Dow Trenchcoat). The asphalt paving showed excellent adhesion to the polymer coating. The combined asphalt paving and polymer coating performed well in severe abrasive sites. The sites exhibiting various levels of corrosion on the plain galvanized end sections of the culverts also reported satisfactory performance for the polymer coating. The age of inspected pipes was 9-13 years.

Ault (2003) discussed laboratory testing and field evaluations of polymer coated CSP. Laboratory testing comprised of five abrasion tests while field investigation was performed on 44 culverts in eight states. A service life model for polymer coated CSP was created, which included four distinct phases (Figure 147). While it is not possible to put time frame to each of these phases, the field studies indicate that pipes between 6 and 27 years old were still in the polymer degradation phase.

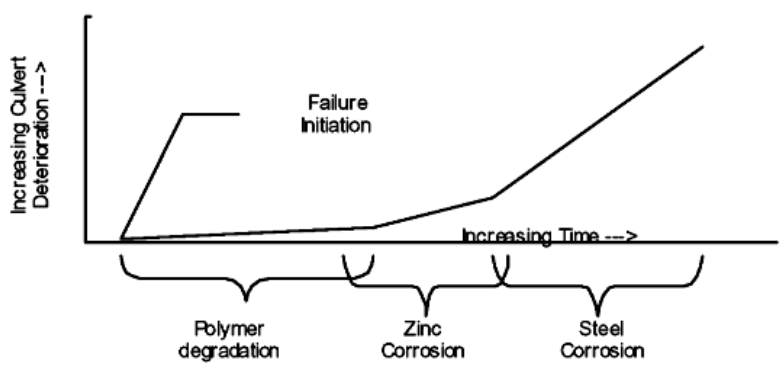


Figure 147. Service life model for polymer coated CSP (Ault, 2003)

Ohio DOT (ODOT) inspected a large number of corrugated metal culverts between 1972 and 1975: 386 structural steel plate pipe (SSP) and 624 corrugated steel pipe (CSP). Of the 624 CSPs, 127 were bituminous coated (AASHTO M 190 Type A) and 302 were bituminous coated with paved inverts (AASHTO M 190 Types B and C). These culverts were nearly all 42 in. diameter or of larger size. The collected data at each site included pipe size, material type, and wall thickness; type of pipe protection; depth and velocity of dry weather flow; presence of abrasive material and apparent effect; amount and

type of sediment or debris or both; pH of water, streambed and embankment; electric resistivity of water, streambed and embankment; description of protection and protection rating; description of base pipe and base pipe rating; qualitative chemical tests; and, metal cores. Detailed analyses were performed to evaluate the effects of various environmental factors on the durability of concrete pipe, galvanized corrugated steel pipe, and bituminous protection of corrugated steel pipe. Equations and graphs were presented to predict the service life of these culvert materials. The study indicated that the environmental conditions in Ohio were aggressive compared to most other states (a large area in Ohio is characterized by non-neutral pH flow and abrasive geological materials). Corrugated metal culverts were shown to be susceptible to corrosion and abrasion, depending on the type of coating and service conditions. Corrosive actions intensify under soil conditions with low pH, low resistivity, and increased moisture and temperature. Abrasive actions amplify with increased drainage flow velocities and coarser, heavier bed loads. Thermoplastic culverts were found to provide higher levels of corrosion and abrasion resistance (Meacham et al., 1982).

Temple et al. (1985) investigated the performance of coated and uncoated galvanized steel and aluminum drainage pipes in Louisiana. One pair of each type of culvert was installed at 10 site locations in 1973. Test sites were selected on the basis of pH and electrical resistivity of the soil. Every two years, one designated culvert of each of the pairs was removed and subjectively rated by a panel. The study showed that the best resistance to corrosion at the majority of test sites had (1) a 14-gage asbestos-bonded asphalt coated galvanized steel pipe and (2) 16-gage galvanized steel pipe with a 12-mil (0.30-mm) interior and 5-mil (0.13-mm) exterior polyethylene coating.

2.5.6. ASSET MANAGEMENT

2.5.6.1 Cost/Risk Analysis

Based on survey of US states and Canada, Perrin and Jhaveri (2004) indicated that very few agencies (3 out of 25 responding agencies) apply some sort of life cycle cost analysis (LCCA), while the majority (15 out of 25 responding agencies) document failures on a cursory or memory basis. A method was developed to compute the total cost of installing a culvert over a given design life, usually 100 years. The total cost is the sum of installation/replacement cost and user delay cost. Several examples of culvert failures were reviewed to illustrate various costs (normal and emergency replacement costs, user delay costs, etc) and demonstrate how longer life would result in significant cost savings in the long-run.

Lian and Yen (2003) performed a comparative study of eight different risk calculation methods on the occurrence probability of inadequate capacity of a culvert to pass floods.

The New York State DOT (NYSDOT, 2008) is currently proposing a ranking metric “the performance indicator” for culverts screening and prioritizing needs. This parameter is calculated not only from the items directly related to the condition of culverts but also elements from the channel rating, thus including a risk element (risks associated with large culverts can be safety risks, e.g., structural collapse, sinkholes, etc, or operational risks, e.g., roads overtopping during storm events, inundation of upstream facilities due to backwater effects, etc). However, a performance target (Table 22) based on culvert condition rating and culvert operational condition (the capacity to carry a storm of given return period with a given headwater Hw to diameter D ratio) remains an important indicator of structural safety, remaining life, and a proxy for the value of the structure as a mean for evaluating the system management performance. Tracking the investment metric using the average condition rating is proposed, as relative trends over time would indicate the effectiveness of various capital investment strategies.

Table 22. Large culvert performance target definitions (NYSDOT, 2008)

Performance	Operational condition	Condition rating
Good	50 yr storm capacity, $H_w/D \leq 1.5$	Structure general recommendation ≥ 6
Acceptable	50 yr storm capacity, $H_w/D \leq 1.5$	Structure general recommendation 5
Deficient	25-50 yr storm capacity, $H_w/D \leq 1.5$	Structure general recommendation 4
Deteriorated	<25 yr storm capacity, $H_w/D \leq 1.5$	Structure general recommendation ≤ 3

3. CULVERT REHABILITATION

3.1. METHODS DESCRIPTION

3.1.1. SLIPLINING

3.1.1.1 Overview

Sliplining is a method of rehabilitation in which a new pipe of smaller diameter is inserted directly into the deteriorated culvert by pulling or pushing. Any pipe type used to construct new culverts can also be used for sliplining (Ballinger and Drake, 1995). With this method, the annular space between the host pipe and the newly installed pipe is created, which is typically grouted with a cementitious material.

The method can be categorized into:

- ***Segmental sliplining*** – A liner is being assembled from short pipe segments at the entry point into the existing pipe, from where the liner is being pulled/pushed into the pipe for the length of each added segment.
- ***Continuous sliplining*** - A liner is manufactured as a continuous pipe or assembled in the field prior to insertion (e.g. by fusing HDPE pipes) to match the entire length of the existing pipe.

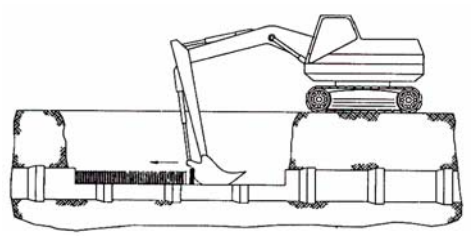


Figure 148. Segmental sliplining (ASTM F 585).

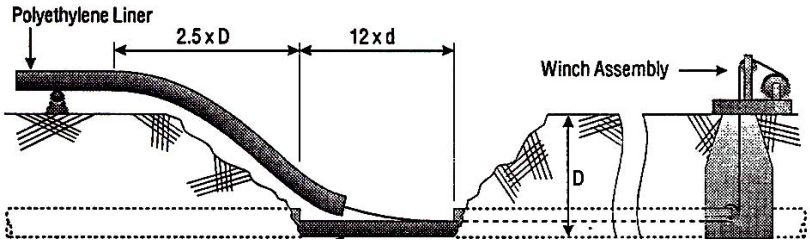


Figure 149. Continuous sliplining (Iseley and Najafi, 1995)

3.1.1.2 Liner Materials

A variety of pipe types can be used for sliplining, e.g. reinforced concrete pipe (RCP), plastic pipes (HDPE or PVC, corrugated on the outer surface of the pipe, corrugated on the outside and inside surfaces, or with both surfaces smooth), corrugated metal pipe, and centrifugally cast glass-fiber-reinforced polymer mortar (CCFRPM) pipes (Ballinger and Drake, 1995).

Corrugated metal pipe, pipe arches and structural plate products are available in many sizes for sliplining culvert pipes. In addition, concrete-lined corrugated metal pipes (CLCMP) can be used, for which the concrete lining is plant-applied (Rinker Materials, 1992).

Johnson and Zollars (1992) investigated three different types of PE pipe for sliplining: smooth PE with mechanical joints, smooth PE with fused joints, and corrugated PE pipe. Newton (1999) discussed the practicality of continuous sliplining of failing culverts with polyethylene (PE) pipe showing that some of the major railroads installed this type of liner in the 1990s with very positive results, and described a coupling system for jointing HDPE pipes by “screwing together” bell-and-spigot ends.

CCFRPM pipes are available in a range of sizes from 18 in. to 110 in. in diameter, and standard pipe lengths of 20 ft. Standard stiffness classes (minimum pipe stiffness in psi) used for sliplining are SN 36

and SN 46 (Hobas Pipe USA, 2008).

One patented system uses stainless steel sleeves for continuous sliplining of culverts (LINK-PIPE, 2008c).

3.1.1.3 Method Applicability

Circular pipes in a wide range of diameters can be rehabilitated with sliplining, i.e., pipes with ID 4 in. to 63 in. can be repaired with continuous sliplining, and 4 in. to 152 in. pipes with segmental sliplining. Custom shapes are possible with segmental sliplining. Diameter changes may prevent this method. Length is not a limitation, as pipelines over 5,000 ft have been sliplined.

The method is typically limited to straight pipe alignment. However, continuous sliplining can accommodate large radius bends.

The method can be applied in any culvert pipe material and shape. Corrugations on the inside surface do not hinder use of this method. Pipe condition is generally not a limitation, e.g., the pipe can be corroded, deformed, and near collapse.

Sliplining can be performed in live flow conditions and flow bypass is seldom required (Thornton et al, 2005).

3.1.1.4 Construction Issues

3.1.1.4_1 Installation Procedure

The following are steps that are normally required for the sliplining process (modified from Thornton et al., 2005, and Ballinger and Drake, 1995):

- Inspect the culvert (diameter changes along the culvert barrel, connecting pipes, protrusions, roots, sediment)
- Determine diameter of sliplining pipe and material
- Clean and clear the culvert, if required
- Divert and/or control water passing through the culvert (setup flow bypass), if required
- Make excavations if required
- Make any repairs in the existing culvert structure that may be necessary prior to sliplining. Repair embankment as well by identifying voids and grouting behind the culvert.
- Construct a guideway on the invert of the culvert, if required to facilitate the sliplining of sections into the existing culvert.
- Install pipe segments or continuous liner into the host culvert pipe
- Upon completion, a 24-hours relaxation period is recommended
- Inspect the installed slipliner pipe (CCTV or man-entry visual inspection)
- Perform leakage or other testing, as required
- Reconnect and stabilize terminal connections. Grout the annular space.
- Restore flow (remove bypass pumping) if applicable, and perform the site cleanup.
- As necessary, complete the project by constructing or modifying head- and wing-walls on the ends of the culvert.

If corrugated pipes or pipe arch sections are used for sliplining of corrugated culverts, either timber skids or a concrete “sidewalk” should be installed in the invert so that the liner may be slid into position. They may not be needed if the culvert is less than 150 ft long and the culvert is 36 in. or less in diameter (Ballinger and Drake, 1995).

When sliplining long culverts or if the insertion is expected to be difficult (e.g., there are offset joints or other irregularities in the existing culvert), conically shaped mechanical pulling heads can be used that enable easier gliding of the liner inside the culvert (Figure 150). For HDPE liners, a less sophisticated but cost-effective approach is to fabricate a pulling head out of a few extra feet of liner (Figure 151). The leading edge of the HDPE liner has evenly spaced wedges cut out, and the remaining ends are collapsed towards the center and fastened together with bolts, threaded rods or metal straps and attached to a cable (Kanters, 2007).

Figure 150. Conically shaped pulling head for sliplining (Kanters, 2007)

Figure 151. Pulling head for sliplining field-fabricated out of the HDPE pipe (Kanters, 2007)

CSPI (2004) outlined steps in relining procedure using corrugated steel pipe and corrugated steel pipe arch.

3.1.1.4_2 Assembling Continuous Liners

Each slipliner product has its own joint design. Heat fusion is a method for joining sections of smooth wall and some corrugated HDPE pipes. After aligning the two ends (Figure 152), pipe sections are connected by welding (Figure 153).

Figure 152. Aligning two pipe sections before welding (D. A. Van Dam, 2008)

Figure 153. Welding HDPE liner sections from inside the pipe (D. A. Van Dam, 2008)

Some HDPE products have integral threads that allow sections to be easily joined without special equipment. A coupling system for joining HDPE pipes by “screwing together” bell-and-spigot is shown in Figure 154. Some HDPE pipes come with a (patented) “snap” joining system (Figure 155) in which two sections of solid wall HDPE pipe, ranging in length from 2 ft to 50 ft and having male and female ends, are aligned and “snapped” together using chains and pressure from the excavator (Snap-Tite, 2008).

Figure 154. Thread-Loc joint for joining HDPE pipes (KWH Pipe, 2008b)

Figure 155. Male end of pipe (left) and a connected joint completed by snapping (right) (ISCO Industries, 2005)

Other types of pipe typically use gasketed or glued bell-and-spigot joints, or a restrained joint mechanism (Figure 156). Figure 157 and Figure 158 show how stainless steel sleeves are assembled into a continuous liner.

Figure 156. "Ring-and-pin" gasketed joint (Potter, 2009).

Figure 157. On-site assembly of continuous liner from stainless steel sleeves (LINK-PIPE, 2008c).

Figure 158. Connecting stainless sleeves into a continuous liner (LINK-PIPE, 2008c).

3.1.1.4_3 Insertion of Slipliner

Duncan (1984) described insertion of a slipliner made from corrugated structural plate arch in one case study where a 240 ft length of failing pipe arch culvert (structural steel plate with span of 14 ft and rise of 9 ft 8 in.) was sliplined with corrugated structural plate arch (10-gauge, span of 11 ft 10 in., rise of 7 ft 3 in). A guide was constructed to help assemble the slipliner pipe from plates and roll it in place. The guide was made from a 1.5 ft × 8 ft channel iron, set to grade every 10 ft and anchored to the culvert every 3.5 ft (Figure 159). A pit in front of the cut-off wall enabled a protective coating (asbestos-bonded asphalt coating) to be manually applied underneath the pipe (Figure 160). The pipe was assembled by bolting together bottom plates first to create the bottom half, and then bolting to it three top plates already assembled on the ground. Rollers were bolted to the pipe every 12 ft (Figure 161). Brackets were installed in the culvert's pipe bottom and the walls, and the slipliner pipe was pulled through the culvert.

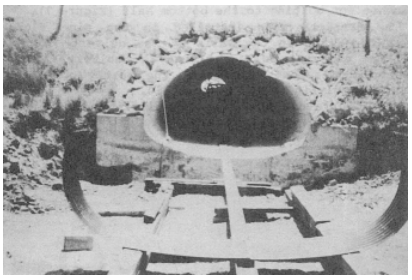


Figure 159. Guide for rolling (Duncan, 1984)

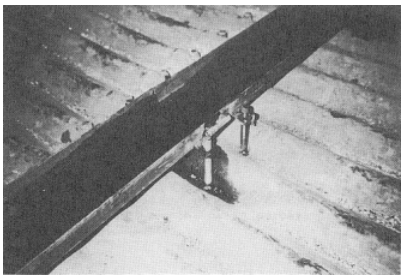


Figure 160. Assembly area and channel guide in place (Duncan, 1984)

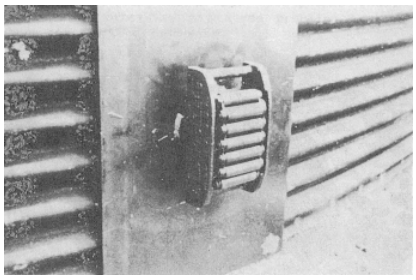


Figure 161. Roller system (Duncan, 1984)

3.1.1.5 Grouting of Annular Space

In sliplining installations, grouting the annulus between the liner and host pipe provides support to the liner pipe that comes from the host pipe that has settled well in the soil. Grouting can increase the structural differential-pressure capability of the polyethylene pipe by up to four-fold. (Phillips Driscopipe Inc, 2002.).

Jenkins and Kroll (1981) studied soil and cement encapsulation of PE pipe to determine the benefit derived in buckling resistance. They showed that cement grout can improve capability of the pipe by a factor of five. Buckling resistance of pipe contained by compacted soil was found 2-3 times greater than that of free standing pipe resistance. They also pointed out that when the annual space is not completely filled with grout, even if the liner pipe cannot generally deform even under much larger loads, the local distortion of the pipe still occurs at the loading point. For that reason, the grout support must be complete, i.e., at least 80% of the circumference must be grouted in order to achieve a higher buckling resistance. When undesirable point loads are avoided in a grouted pipe, the grout in the annulus protects the liner pipe and thus enhances the durability of the pipe.

Zhao et al. (2003) presented the effect of the grout on the performance of slipline pipe based on a three-year study on the performance of a sliplined water main in the City of Ottawa, Canada (Table 23). They also stated that the decision on annular space grouting is mainly based on its impact on construction and cost. Grouting the annulus also minimizes the buckling potential due to water accumulation in the annulus and freeze-up when the pipe is installed within the frost susceptible depth in cold regions.

Table 23: Pros and cons of grouting the annular space in sliplining (modified after Zhao et al, 2003)

Advantages of using the grout	Disadvantages of using the grout
▪ Increases the buckling resistance of the liner pipe	▪ Increased construction cost and longer installation time
▪ Increases the load carrying capacity	▪ Potential collapse of liner pipe during grout injection
▪ Can be used to control load-sharing between liner and host pipe	▪ Requirement for blocking of all openings that may allow grout to escape during pouring
▪ Eliminates sharp loading edges on the liner pipe from failed host pipe	▪ Requirement for a proper grout injection procedure
▪ Reduces longitudinal movements due to differential temperatures, thus minimizing shear-off potentials at lateral connections	
▪ Increases the service life	

Duncan (1984) provided recommendations on how to position the holes for grouting the annular space: at 3 and 9 o'clock positions at 8 ft intervals along the sliplining pipe, and at 11 and 1 o'clock positions at 4 ft intervals. Also, to facilitate filling under the bottom of the arch culvert, grout holes should be at 5 and 7 o'clock positions at 4 ft intervals.

Grouting of the annular space between the pipes must be continuous with no voids (a few small voids can be tolerated) to achieve the improved structural support that is needed. The Phillips DriscoPipe (1991) design manual showed the importance of complete grouting for sliplining with plastic pipe. The maximum external pressure differential on unsupported liner will increase when the gap between the liner and the original culvert is eliminated. For example, based on liner design theory at that time, Phillips DriscoPipe (1991) indicated that service life of ungrouted HDPE SDR 26 pipe is 50 years for 9 ft of water head. However, when the same pipe is properly grouted into an existing pipe, it can withstand 36 ft of external groundwater.

Stephens (1996) provided useful guidelines for grouting the annular space of sliplined pipe (test shown was modified by authors):

- The grout design should specify mix design (proportions of constituents), density of slurry (cement, cement/flyash and water) and density of grout (after dispersant is added), viscosity, initial set time,

24-hour and 28-day compressive strengths, shrinkage, stability, and "bleed" (fluid loss).

- Initial set-up time is extremely important. The grout mix must remain fluid and not thicken for a period of at least two hours (see Figure 164). Grout should be tested in accordance with ASTM C939.
- Fly ash based lightweight grouts are not recommended for slipliner grouting if the grout would be exposed to excessive water infiltration before it sets.
- If the existing pipe has deflected from a straight alignment, there is the possibility that trapped air will result in discontinuous grout.
- Grout injection should start at the upstream end of the pipe and progress toward the downstream end so as to more easily displace water and debris. Suitable injection tubes must be inserted at the upstream end (e.g, as shown in Figure 162). Vent pipes installed at the downstream end should be 150% larger than injection tubes to minimize the potential for clogging.
- In the field, every batch of grout should be tested for density and viscosity.
- Any suspected voids in the soil must be pressure grouted prior to inserting the slipliner.
- Maximum grout injection pressure must not exceed the slipliner manufacturer's recommendations.

For evaluating the proposed grout mix, the following need to be considered (Stephens, 1996): (1) condition of the pipe wall beneath the corroded layer, (2) groundwater infiltration; (3) porosity of the corroded portion of pipe (for fluid loss considerations, see Figure 163), (4) the length of runs, type of pipe to be used, OD of slipliner pipe, pipe stiffness, maximum injection pressure allowed, weight of pipe filled with water (if flotation or misalignment are not allowed), and (5) the joint deflection limit of the slipliner pipe if flotation is expected.

Figure 162. Grout tubes and slipliner pipe inside existing culvert (D. A. Van Dam, 2008)

Figure 163. A corroded pipe can cause thickening of the grout due to excessive fluid loss (Stephens, 1996)

Figure 164. A collapsed 18 in. welded steel pipe in a 36 in. steel casing attributed to too quick grout set-up time (Stephens, 1996)

3.1.1.6 Other issues

3.1.1.6_1 Flammability of Polyethylene (PE) Slipliners

North Dakota DOT incurred severe damage to some polyethylene (PE) liners installed in corrugated metal culverts due to ditch fires. Katti et al. (2003) investigated options to address the flammability of these liners. The research reviewed several coatings that could be applied to the inside of PE liners, as well as CIP liners and centrifugally cast glass-fiber-reinforced polymer mortar (CCFRPM) pipes. The CCFRPM pipe was found to be the best solution, provided it was fitted with concrete end caps to ensure resistance to fire.

3.1.1.7 QA/QC Considerations

PPI (1995) prepared *Guidance and Recommendations on the Use of Polyethylene (PE) Pipe for the Sliplining of Sewers,* which describe the specifications, design considerations, and installation procedures for continuous sliplining utilizing polyethylene liners.

Goodwin et al. (1997) provided guidelines for writing specifications for successful sliplining.

Indiana DOT's standard specifications (INDOT, 2009) require a Quality Control Plan (QCP) to be submitted to the engineer for acceptance at least 15 days prior to the start of sliplining, which must include, as a minimum, identification of the QC representative by name and documentation verifying the QC representative's experience; the contractor's method for cleaning and preparation of the existing pipe; method for joining, welding, or fusing the pipe joints; the personnel who will be welding or fusing the liners and their certification; the method and frequency of destructive and non-destructive testing on the welded or fused joints; the initial testing of the first joint, weld or fusion at each liner installation location; the corrective action that will be taken if defective or non-passing joints are found; the grouting process including the daily calibration process procedures for the grout generating equipment; inspection of bulkheads; specific job mix of the grout concentrate; the grouting procedure to ensure complete filling of voids; the corrective action to be taken if the grout compressive strength does not meet specifications; and the corrective action if the installation of the grout causes damage or deflection to the liner.

As grouting work is typically sub-contacted and the quality of grouting contractors can vary considerably, Caltrans (2003) recommends a list of submittals and calculations that the grouting sub-contractor should be required to forward to the project engineer.

3.1.1.8 Standards and Specifications

ASTM D2657 describes the general procedures for making joints with polyolefin pipe and fittings by means of heat fusion joining techniques (material standard, continuous sliplining).

ASTM D3212 covers joints for plastic pipe systems intended for drain and gravity sewage pipe at internal or external pressure less than 25 ft head using flexible watertight elastomeric seals. Test requirements, test methods, and acceptable materials are specified (material standard, segmental sliplining).

ASTM D3262 covers machine-made glass-fiber-reinforced thermosetting-resin (fiberglass) pipes (Material standard).

ASTM D4161 covers axially unrestrained bell-and-spigot gasket joints including couplings required for machine-made "fiberglass" (glass-fiber-reinforced thermosetting-resin) pipe systems, 8 in. through 144 in. (material standard, segmental sliplining)

ASTM D6783 covers the testing and requirements for polymer concrete pipes. This specification is suited primarily for pipes to be installed by direct burial and pipe jacking but may be for sliplining as well (material standard, segmental sliplining).

ASTM F585 describes the design considerations, material selection considerations, and installation procedures for the rehabilitation of sanitary and storm sewers by the insertion of polyethylene pipe through the existing pipe, along the previously existing line and grade (installation standard).

ISO/TR 10465-1 describes the procedures for underground installation of flexible GRP pipes (Installation standard).

NASTT (2006f) outlined QA/QC issues for thermoset pipe (e.g., CCFRPM) that can be installed by sliplining. QA/QC should address the component products (the resin, the fillers and the reinforcing agents), the design (thickness, host pipe configuration, corrosion resistance, hoop strength and fit), and installation (joint fit, installation method, lateral restoration and grouting).

Additional standards and specifications associated with sliplining are listed in Thornton et al. (2005).

3.1.1.9 Example Case Histories

3.1.1.9_1 Sliplining with RCP

Segmental sliplining of a twin barrel metal culvert with precast concrete pipe in Woodbridge, VA is illustrated in the appendix of FHWA's *Culvert Repair Practices Manual* (Ballinger and Drake, 1995). The host corrugated metal pipe (CMP) was a 96 in. vertical oval pipe. After a concrete slab was cast to fill erosion holes under the invert and to re-establish the grade, the 66 in. concrete pipe sections were installed, and the annular space was grouted.

Vaillancourt (2002) described segmental sliplining with reinforced concrete pipe (RCP) for the Maine DOT in Lewiston, ME. The existing corrugated metal culvert, 1,048 ft long and 144 in. in diameter, was relined with a 108 in. RCP structure. The RCP segments were installed by "pushing-in-place" the segments inside the CMP. A special cart was fabricated to drive the precast concrete segments inside the pipe. Jacks were used to lift the segments off the ground while the cart was pushed along the existing tunnel with a Bobcat loader. At the target location, the jacks were lowered and the pipe homed with the previously positioned pipe using two 6-ton come-alongs anchored in two holes that were later used for pumping grout between the old metal and new concrete pipes. The process was repeated for each pipe segment.

3.1.1.9_2 Sliplining with CMP

In Pittsburgh, PA, Pennsylvania DOT sliplined a failing 48 in. reinforced concrete pipe with a corrugated steel pipe under a height of cover of 91 ft (Project SR6060 18B). Grout couplings were furnished to facilitate concrete grout placement between the new corrugated steel pipe and the existing concrete pipe (CONTECH, 2009b).

In Carroll County, MO, Missouri DOT has sliplined an existing concrete box culvert (twin 12 ft x12 ft culverts) on Route 65 using a 10 gage, 10 ft diameter Aluminized Steel Type 2 corrugated steel pipe (CONTECH, 2009b).

In Evergeen, CO, an existing 96 in. culvert with a deteriorating invert, 44 ft long, was sliplined using a 12 gage, 78 in. in diameter, Aluminized Steel Type 2 pipe (Figure 166). The annulus was grouted with flowable fill.

Figure 165. Sliplining of concrete box culvert (CONTECH, 2002)

Figure 166. Sliplining of existing corrugated metal culvert (CONTECH, 2009a).

Duncan (1984) reported the use of corrugated structural plate arch for sliplining a corroded structural steel plate culvert by Montana DOT. In a case study near Hardin, MT, the 260 ft long deteriorated culvert with a cross-section of 14ft × 9ft 8 in. was sliplined.

Minnesota DOT sliplined a failing concrete box culvert (double box) with corrugated metal pipe arch, 112 in.×75 in., 12 gage, on two 100 ft long runs (Figure 165). The project was completed in the summer of 2002 (CONTECH, 2002; Larry Randall, Mn/DOT, personal communication).

3.1.1.9_3 Sliplining with HDPE

Selected culvert rehabilitation projects utilizing large diameter, lightweight HDPE pipe have been summarized in case histories downloadable from the web (Table 24). The HDPE pipe has smooth outer wall for easy insertion and smooth inner surface. It is available in a wide range of sizes from 18 in. to 120 in. in diameter, and standard pipe lengths of 20 ft, 40 ft and 50 ft. The slipliner pipe is typically delivered to site in sections and after aligning the pieces are welded from the inside of pipe. After completed welding, the pipe is pushed into the existing culvert, and the annular space grouted (KWH Pipe, 2008b).

Table 24. Selected culvert sliplining projects with large diameter, lightweight HDPE pipe

Project	Existing culvert			Slipliner pipe	Reference
South Indiana, I-64	CSP	96 in. ID		HDPE 72 in. OD	(KWH Pipe, 2008a) case study #110
HWY 401, Canada	CMP	24 in. ID	300 ft	HDPE 21 in. OD	(KWH Pipe, 2008a) case study #111
Falmouth, ME	CMP	90 in. ID	122 ft	HDPE 90 in. OD	(KWH Pipe, 2008a) case study #117
Bancroft, Ontario	CSP	77 in. ID		HDPE 59 in. OD	(KWH Pipe, 2008a) case study #124
Veil, CO, I-70	CMP		235 ft	HDPE 66 in. OD	(KWH Pipe, 2008a) case study #136
ODOT District 12, I-90	CMP	102 in. ID	835 ft	HDPE 72 in. OD	(D. A. Van Dam & Associates, 2008)
ODOT District 5, I-70	CMP	108 in. ID	322 ft	HDPE 84 in. OD	(D. A. Van Dam & Associates, 2008)
Crawford County, OH	CMP	108 in. ID	120 ft	HDPE 90 in. OD	(D. A. Van Dam & Associates, 2008)
Williams County, OH	CMP	54 in. ID	180 ft	HDPE 42 in. OD	(D. A. Van Dam & Associates, 2008)
Summit County, OH	CMP	102 in. ID	123 ft	HDPE 90 in. OD	(D. A. Van Dam & Associates, 2008)
ODOT District 7, OH	CMP		325 ft	HDPE 54 in. OD	(D. A. Van Dam & Associates, 2008)
ODOT District 10, OH	CMP	96 in. ID	143 ft	HDPE 72 in. OD	(D. A. Van Dam & Associates, 2008)
Cuyahoga County, OH	CMP	120 in. ID	190 ft	HDPE 96 in. OD	(KWH Pipe, 2008a) case study #126; (D. A. Van Dam & Associates, 2005)
ODOT District 10, Columbus, OH, I-270	CMP	156 in. ID	440 ft	HDPE 119 in. OD	(D. A. Van Dam & Associates, 2006)

Selected culvert rehabilitation projects utilizing HDPE pipes "snap" joining system have been summarized in field reports downloadable from the internet (Table 25). Examples of culverts sliplined using a "snap" jointing system are shown in Figure and Figure 168.

Figure 167. Snap-Tite® inside the CMP culvert (ISCO Industries, 2005)

Figure 168. Snap-Tite® inside the concrete box culvert (ISCO Industries, 2005)

Table 25. Selected culvert sliplining projects utilizing HDPE pipes with "snap" joining system

Location	Year	Existing culvert			Slipliner pipe	Reference
Bartow, WV	2002	CMP	30 in. ID	116 ft	HDPE 20 in. OD	(Snap-Tite, 2008)
Marion County, OR	2001	CMP	30 in. ID	110 ft	HDPE 28 in. OD	(Pech, 2001)
Monmouth, NJ	2001	CMP	54 in. ID	475 ft	HDPE 48 in. OD	(Snap-Tite, 2008)
Ruth Lake, CA	2000	CMP	72 in. ID	190 ft	HDPE 63 in. OD	(Snap-Tite, 2008)
Weston, WV	2008	CMP	66 in. ID	156 ft	HDPE 54 in. OD	(Snap-Tite, 2008)
Bay County, MI	2006	Concrete Pipe	74 in. ID	50 ft	HDPE 72 in. OD	(Snap-Tite, 2008)

Pech (2001) described another sliplining case study in Marion County, OR. A 30 in. CMP culvert, 110 ft long and 20 ft below the road surface, was sliplined with 28 in. HDPE pipes with “snap” joining system. The liner was installed in 20 ft segments. The push was rather difficult (all the tools used were a backhoe, chain, and a couple come-alongs, and the push was carried out from an awkward angle to preserve as much of the ditch bottom as possible) but completed successfully. Having liner with a degree of flexibility proved advantageous (the existing CMP had developed a bit of a dog leg over the years, and the operator with some effort was able to push the liner past that point). Smooth joint transitions reduced the possibility of debris getting caught and creating obstructions. No grouting of annular space was performed because the 28 in. liner fitted snugly inside the existing CMP, however, the crew returned to pour concrete head walls the following day.

The FHWA completed a culvert rehabilitation project in Shenandoah National Park, VA in 1999 (ISCO Industries, 2001). The project scope included sliplining of 56 CMP culverts with inside diameters ranging from 18 in. to 36 in., with total length of 4,800 ft. The slipliners were HDPE pipes with “snap” joining system. The project was unusual because traversing the Appalachian Mountains required installation of pipe on slopes as steep as 60°.

Oregon DOT reported sliplining corrugated metal culverts with HDPE pipe. A 36 in. diameter and 200 ft long pipe (the Harbor Hole Culvert Rehab Project) was sliplined with 34 in. HDPE pipe. The annular space was subsequently grouted (John Woodroof, ORDOT, personal communication).

Two segmental sliplining projects were completed in Iowa in 1995. In Audubon County, IA, a 102 ft long 36 in. deteriorated corrugated metal culvert was sliplined with a 32 in. HDPE pipe. In Hamilton County, IA, a 339 ft long 42 in. culvert was sliplined with a 32 in. HDPE pipe (Technology News, 1997).

3.1.1.9_4 Sliplining with PVC

New Hampshire DOT District 2 has been actively sliplining culverts if there is deep cover or high traffic volume. It reported, for instance, sliplining of two culverts with PVC pipes (UNH T2, 2006). One of these culverts, a 48 in. concrete culvert, 100 long and 25 ft deep, was sliplined with a 36 in. liner because the existing pipe was no longer straight and could not accommodate a larger diameter liner.

3.1.1.9_5 Sliplining with CCFRPM

Selected culvert rehabilitation projects utilizing centrifugally cast glass-fiber-reinforced polymer mortar (CCFRPM) pipes are summarized in Table 26. CCFRPM pipes are available in a range of sizes from 18 in. to 110 in. in diameter, and standard pipe lengths of 20 ft. Standard stiffness classes (minimum pipe stiffness in psi) used for sliplining are SN 36 and SN 46 (Rick Turkopp, Hobas Pipe USA, personal communication).

Table 26. Selected recent culvert sliplining projects utilizing CCFRPM pipe

Location	Year	Length	Slipliner pipe
Houston, TX	2007	60 ft	CCFRPM pipe ID 24 in.
Smyrna, GA	2007	417 ft	CCFRPM pipe ID 72 in.
Smyrna, GA	2007	1,143 ft	CCFRPM pipe ID 78 in.
Smyrna, GA	2007	39 ft	CCFRPM pipe ID 84 in.
Reading, PA	2006	80 ft	CCFRPM pipe ID 30 in.
Orlando, FL	2006	80 ft	CCFRPM pipe ID 72 in.
Norristown, PA	2006	112 ft	CCFRPM pipe ID 72 in.
Pasadena, TX	2005	3,945 ft	CCFRPM pipe ID 18 in.
Pasadena, TX	2005	1,701 ft	CCFRPM pipe ID 24 in.
San Antonio, TX	2005	2,666 ft	CCFRPM pipe ID 42 in.
Houston, TX	2005	31 ft	CCFRPM pipe ID 42 in.
Fayetteville, NC	2005	100 ft	CCFRPM pipe ID 54 in.
Boston, MA	2005	292 ft	CCFRPM pipe ID 96 in.

3.1.1.9_6 Sliplining with Stainless Steel

Under Highway 115/35 in Canada, a corrugated metal culvert, 156 ft long and 39 in. in diameter, was repaired with a liner assembled from short stainless steel sleeves in 2007. The sleeves used were 36 in. in diameter (LINK-PIPE, 2008b).

Oregon DOT has sliplined a 105 ft long wooden culvert under Hwy 273 in 2009. The culvert installed in 1939, was made from creosoted timbers assembled to form a tunnel 4.5 ft wide on the bottom, 3.5 ft wide on top, and 6.5 ft high; the floor of this culvert was made of supported and continuous creosoted 8×12's. The tunnel collapsed forming a slowly draining 67 ft deep lake on one side of the highway. Rehabilitation was needed to allay concerns that this section of highway might slide down a steep slope and bury the I-5 freeway nearby. A new 30 in. steel casing was installed along the tunnel floor using a tunnel jacking and boring machine (Figure 169). At the end of the culvert, a standpipe (a 30 in. CMP vertical pipe, 45 ft deep) was located (Figure 170). The pipe was CIP relined (George Vernon, Mill Creek Management Technologies, personal communication).

Figure 169. Sliplining with steel casing using a tunnel jacking and boring machine (George Vernon, personal communication)

Figure 170. A 45 ft deep standpipe at the end of culvert was CIP relined (George Vernon, personal communication)

3.1.1.10 Advantages and Limitations

The main advantages of sliplining are simple installation, the ability to rehabilitate practically any pipe size, the variety of sliplining pipes on the market, and no need for flow bypassing in most cases. Thus, sliplining often offers an economical rehabilitation option for culverts. The method is capable of accommodating large radius bends. The method does not involve chemical processes and may be environmentally safe relative to other procedures (there is no Styrene, disposal off potentially

contaminated process waters).

The main limitations of sliplining are the need for excavation of pits (although with shorter culvert lengths, digging of access pits may be avoided), and the grouting of the annular space (which is generally required). Other limitations often quoted are the reduction in flow cross-sectional area (although flow capacity could be recovered, or even increased, due to smooth interior surface of slipliner pipe) and the need for a sufficient work area (this can be significant). Numerous joints can be created with segmental sliplining, whereas with continuous sliplining the number of joints can be limited to only few.

3.1.2. SPIRALLY WOUND LINERS

3.1.2.1 Overview

Spirally wound liners are fabricated in the field from a continuous thermoplastic strip that has one male and one female edge. During the helical winding process, the male and female edges self-interlock forming a leak tight joint (Figure 171).

Figure 171. Self-interlocking of male and female edges during helical spiral winding (Jakovac and Raz, 2003)

The process of spiral winding for creating the liner inside the pipe is also utilized by grout-in-place liners (GIPL), which are covered in Section 3.1.6. GIPL liners use however a high-strength structural grout in the annular space between the liner and the host pipe, whereas spirally wound liners covered in this section use non-structural grout or do not require grouting the annular space at all.

3.1.2.2 Materials Used

Strips for winding can be made of either PVC or HDPE. The strips come in a variety of profiles (Figure 172) with external ribs to increase the liner stiffness and to anchor the liner in the cement grout if used for annular space grouting. The PVC strip can be made with steel reinforcement (Figure 173), though this is typically used for larger diameter liners, e.g., 30 in. or more (Jakovac and Raz, 2003).

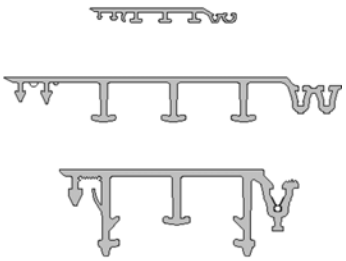


Figure 172. Some examples of winding strip profiles (from Jakovac and Raz, 2003)

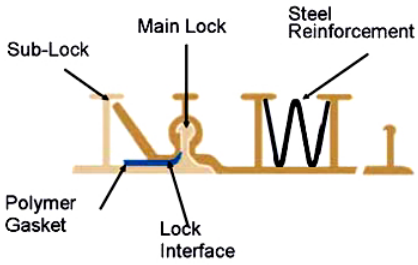


Figure 173. A PVC profile with steel reinforcement (Doherty, 2006)

Based on the location of the winding machine during installation, spiral winding can be classified as either:

- Stationary – A winding machine is positioned at an opened end of the culvert pipe (Figure 174). The machine is a fabricating assembly that consists of a drive tray and various winding cages.
- Mobile – A winding machine traverses the pipe. The machine is circular in shape consisting of a series of hydraulic rams that project radially from the central hub (Figure 175).

Figure 174. A stationary winding machine (Jakovac and Raz, 2003)

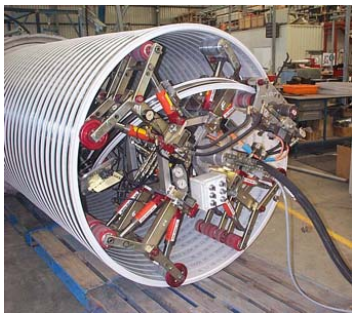


Figure 175. A rotating winding machine that traverses through the pipeline (Jakovac and Raz, 2003)

This method may or may not create intimate contact between the liner and the host pipe:

- Fixed diameter liners – Most spiral winding systems create an annular space between the host pipe and the liner, which is grouted, generally using cementitious grouts (Figure 176). The final result is a composite made of spirally wound liner, grout and the host pipe.
- Tight fitting liners (expandable diameter liners) – In a two-step installation process, the liner is first wound into the existing culvert pipe at a diameter smaller than the existing pipe and subsequently mechanically expanded to create a tight fit within the culvert pipe that eliminates the need for grouting (Figure 177). See Section 3.3.3 for additional details.

Figure 176. Fixed diameter liner spirally wound inside a CMP culvert before (left) and after (right) grouting of annular space (Jacquie Jaques, CPT USA, personal communication).

Figure 177. Tight fitting liner is initially wound into the culvert at diameter smaller than the existing culvert pipe (left) and subsequently expanded to closely fit the existing culvert pipe (Jakovac and Raz, 2003)

3.1.2.3 Method Applicability

Spiral winding can rehabilitate circular pipes with diameters from 6 in. to 180 in. (Jaques, 2008a). The maximum drive length is dependent on several variables and project specifics, and is considered to be limited to 650 ft (Sterling et al., 2009). Jacquie Jaques (CPT USA, personal communication) reported shorter maximum drive lengths: up to about 440 ft for fixed diameter liners, about 210 ft for expanded liners, and 175 ft for “close fit” liners installed with a winding machine that traverses the pipeline.

Applicability is not limited by culvert pipe type, shape, corrugations on the inside surface, or condition. Installation can be performed in live flow conditions.

3.1.2.4 Construction Issues

3.1.2.4_1 Installation Procedure

Thornton et al. (2005) outlined the general installation procedure, which involves the following steps:

- Test the air in the pits for the presence of toxic or flammable vapors.
- Clean the existing culvert (e.g., using high-velocity jet cleaners).
- Inspect culverts for protrusions, collapsed or sagged sections, etc, that may hinder liner installation and remove any obstructions found.
- If necessary, setup flow bypass.
- If required or recommended, excavate an insertion pit.
- Position the winding machine (e.g., within the insertion pit) so that the liner can be wound directly into the culvert. Wind the liner placing the required sealant or adhesive within the primary and secondary locks of the locking configuration at the edge of the strip (unless already in place).
- Expand the liner, if required.
- If the job requires profile strips in the form of panels, cut and trim the panels to fit as near as practical to the internal diameter of the existing culvert or to produce the required annulus. Place the panels square with the culvert wall, circumferentially, and lock adjacent panels together as specified by the manufacturer. Seal termination joints with a manufacturer-supplied connecter and approved sealant.
- Inspect the completed installation (CCTV or visually in man-entry pipes).
- Carry out leakage or other testing, if required
- Reconnect connections (if applicable).
- Inject grout into the annular space between the existing culvert and liner through openings in the end seals, at reconnected service connections, or through holes drilled into the liner at appropriate points. Carry out the grouting procedure (apply the grout in a series of lifts/stages or apply the grout continuously)
- Restore flow if bypass was required and initiate site cleanup.

3.1.2.4_2 Diameter Expansion

Jakovac and Raz (2003) provided detailed descriptions of how spiral winding can produce tight-fit liners. In a two-step installation process, the liner is first wound out into the existing culvert pipe at a diameter smaller than the existing pipe and is subsequently expanded to fit the existing culvert pipe tightly (Figure 178).

For this purpose, a PVC profile is designed with two different types of locks:

- A secondary lock (an assembly lock) prevents the pipe from fully expanding during winding (this is a sacrificial lock that will be opened during the subsequent expansion process)
- A primary lock (a main lock) guides the expansion process and later, together with an adhesive, ensures the tightness of the relined pipe.

This PVC profile also has a hot melt adhesive that is factory applied to the secondary lock (shown blue in Figure 179, Figure 180) and a two-component joint lubricant sealant (silicone or polyurethane) that is field applied to the zone around the main lock (shown green in Figure 179, Figure 180) to act as a lubricant during the expansion process.

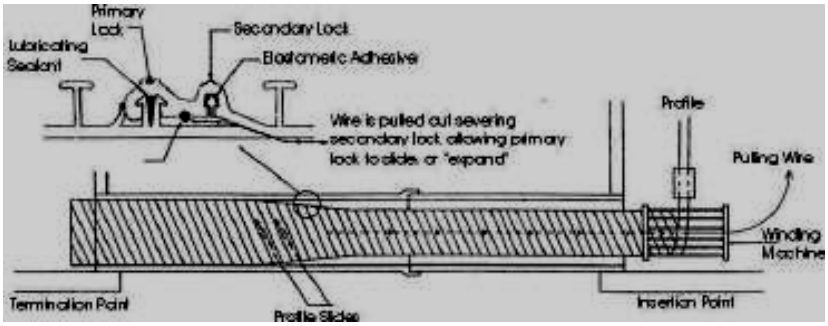


Figure 178. Schematic of spiral-winding process with diameter expansion (PRS Rohrsanierung GmbH, 2008)

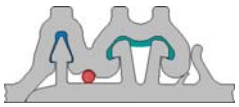


Figure 179. Profile lock before expansion showing wire (red circle) and a sacrificial lock intact (Jakovac and Raz, 2003).

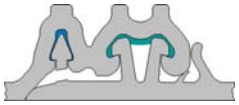


Figure 180. Profile lock after expansion showing no wire and the sacrificial lock opened (Jakovac and Raz, 2003).

The profile is wound from an "entry end" to an "exit" end of the host culvert pipe, with a high tensile strength steel wire in between primary and secondary lock (red circle in Figure 179). Once the spiral liner pipe has reached the exit end of the host pipe, the steel wire is pulled out. This uncouples the secondary lock and allows the liner pipe to be expanded. Expansion is carried out by fixing the end of the liner pipe (at the exit end) to prevent it from rotating and continuously winding the liner from the other end. This results in a circular pipe that is designed to be structurally self supporting.

3.1.2.4_3 Grouting of Annular Space

With most spiral winding systems, annular space is created between the host pipe and the liner and grouting is required. Generally, non-structural cementitious grout is used, e.g., 300 psi compressive strength, installed in accordance with section 500-3 of the Greenbook (BNI, 2009), because the liner is designed to be structural and the grout is only required to provide a load path from the existing pipe to the liner and not to enhance the strength of the liner. With tight fitting liners, grouting is not required unless: (1) excessive sections of pipe are missing, (2) significant offset of the joints exist in the host culvert pipe (i.e., 12.5% of ID or 1 in., whichever is greater), or (3) pipe ovality exceeds 5% (Jaques, 2008a).

Grouting of the annular space is carried out through grout pipes that are either installed from the surface prior to liner winding (Figure 181) or drilled through the installed liner (Figure 182). Grouting can also be performed through the bulkhead (Figure 183).

Figure 181. Surface grouting through installed grout pipes prior to liner installation (Jaques, 2008a).

Figure 182. Grouting through grout ports drilled through the installed liner (Jaques, 2008a)

Figure 183.Grouting through the bulkhead (Jaques, 2008a)

3.1.2.5 Example Case Histories

Virginia DOT rehabilitated two 60 in. CMP culverts, each 60 ft long, using spiral winding. Each culvert

had significant joint offsets inside the line and was lined using a 52.5 in. ID liner. It took 50 minutes to line each culvert (Bridget Donaldson, Virginia DOT, personal communication).

California DOT (Caltrans) used spiral winding to rehabilitate two culverts under Highway 101 in San Jose, CA, in 2007 (Vitorelo, 2008). The culverts were reinforced concrete pipes, 18 in. and 30 in. in diameter, and with total length of 230 ft. The culverts were broken and leaking, and sags formed in the freeway pavement above the pipes. The 18 in. culvert had a gaping 8 in. hole that was spewing water with a force equal to a 15 ft water head. The spiral winding spanned the hole and sealed the leak.

California DOT (Caltrans) used spiral winding on other culverts as well. For example, a 24 in. culvert made of ribbed plastic and approx 200 ft long (consisting of three pipe segments separated with manholes), located under Highway 4 in Contra Costa County, CA, was relined in 2002 (Jon Vitorelo, Caltrans, personal communication).

County of San Diego, CA, reported utilizing spiral winding to rehabilitate a 36 in. corrugated metal culvert under Honey Springs Road in San Diego, which had been badly deteriorated and was on the verge of collapse (the invert of the culvert had rusted out, and the supporting bedding soil had been eroded away). Prior to the relining, the invert was rebuilt with four tons of stone and then sealed with quickset mortar so that the grout to be pumped into the annulus would not migrate into the stream. The project specified installation of a 33 in. outside diameter liner but due to deformations in the host pipe, the largest fixed diameter liner that could be installed was 30 in. (Rib Loc Australia, 2008).

Gerhardt (1998) reported on several case studies of spiral winding in Europe.

3.1.2.6 QA/QC Considerations

ASCE (2010) outlined QA/QC measures applicable for spiral winding. The profile strip used for winding must be marked on the surface with a code number identifying the manufacturer, plant, date, and profile designation. Structural grout should be sampled and tested as designated by the owner (e.g., compressive strength test, bleed test, shrinkage test, and flowability test). The installed liner should be inspected by direct visual inspection or with CCTV. The manual also outlined acceptance and delivery, and time and cost considerations.

3.1.2.7 Standards and Specifications

ASTM D1784 covers rigid PVC compounds and chlorinated PVC compounds for use in extruded or molded form, including piping applications. (Material standard)

ASTM F1697 and **ASTM F1735** covers the requirements and test methods for materials, dimensions, workmanship, stiffness factor, extrusion quality, and a form of marking for extruded PVC profile strips for machine-made field fabrication of spirally wound pipe liners (Product standards).

ASTM F1698 and **ASTM F1741** describe procedures for the rehabilitation of sewer lines and conduits by the installation of a field-fabricated PVC liner. After installation of the liner, cementitious grout is injected into the annular space between the liner and the existing sewer or conduit (Installation standards)

Additional standards and specifications associated with spirally wound liners are listed in Thornton et al. (2005).

3.1.2.8 Advantages and Limitations

The main advantages of spiral winding are that it eliminates the need for excavation, pipe storage on site, and bypass flow (for most applications). Installation is quick and quiet, and this method has the ability to accommodate large radius bends and diameter changes. The method does not involve chemical processes and is more likely to be environmentally safe (no Styrene and potentially contaminated process waters to dispose off).

The main limitations of spiral winding are the need for grouting of the annular space (unless diameter expansion has been applied), the reduction in flow area (although flow capacity is often recovered or even increased due to smooth interior surface of the liner pipe), and the ends of the relined pipe require watertight sealing. The method is applicable in circular pipes only.

3.1.3. CURED IN PLACE PIPE (CIPP)

3.1.3.1 Overview

Cured-in-place (CIP) relining is a method in which a flexible material (typically a tube) saturated with thermosetting resin is inserted into the deteriorated culvert by inversion or winching, expanded via air or water pressure, and the resin is subsequently cured at ambient or elevated temperature (by means of steam or hot water) or by means of UV-light. The final product, which is often referred to as Cured in Place Pipe (CIPP), has minimal or no annular space, thus eliminating the need for grouting (Figure 184).

The CIP liners can be categorized into conventional CIPP and composite CIPP (based on their material composition, see 3.1.3.2). Composite CIP liners are high-strength fiber-reinforced CIP liners (fiber reinforcement provides increased stiffness and strength resulting in thinner liner walls compared to conventional CIP liners) and are used to rehabilitate medium to large sewers, drains and culverts.

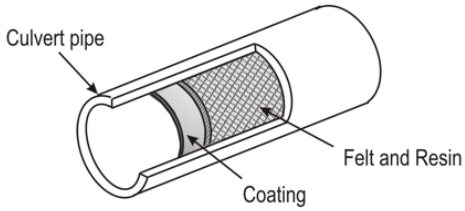


Figure 184. Components of CIP liners

Figure 185. CIP relined culvert (McClanahan, 2009)

3.1.3.2 Materials Used

3.1.3.2_1 Conventional CIPP

In conventional CIP lining, the fabric tube is typically made of needled felt or equivalent woven or non-woven material and a thermosetting resin is usually unsaturated polyester, epoxy vinyl ester, or epoxy with catalysts. Simonson and Peterie (2002) indicated that roughly 95% of CIP liners are made with polyester, 4% with vinyl ester and 1% with epoxy resins. A protective layer of non-porous fabric (reinforced PVC or PE) is often added to the outside of the tube (after installation is completed, it faces the pipe interior if the liner is inserted by inversion or it stays between the CIPP and the host culvert if the liner is pulled into place).

In addition, fiberglass and resin systems are available that are cured with ultraviolet (UV) light. The

most commonly used fiberglass is ECR-glass, which has mechanical properties similar to E-glass but enhanced corrosion resistance (Lance Brown Import-Export, 2009). The resin used in these systems (polyester, vinyl ester, or non-styrene resins) comes with a UV initiator. A UV-cured CIP liner typically consists of several layers: (1) an outer foil, i.e., there would be no bonding to the host pipe, (2) a water resistant layer, (3) the resin impregnated layers of fiberglass, and (4) an inner foil that would be removed after the curing so as not to hinder future cleaning operations, e.g., high-pressure water jetting (Sain and Montemarino, 2009).

3.1.3.2_2 Composite CIPP

Shearer (2007) described one type of composite CIP that utilizes a tube made of conventional felt material sandwiched between layers reinforced with carbon and/or glass fiber liner (Figure 186). The reinforced layers are manufactured by stitching carbon or glass fiber tows[1] onto thin layers of felt (Figure 187). Typically, glass is the material of choice for both layers in the composite CIP and carbon is used when a higher stiffness composite is required or in industrial settings where higher corrosion resistance is needed. In addition, a coating made of thermoplastic material, e.g., polypropylene (PP) is added to the tube for corrosion protection. The resin used is standard isophthalic polyester resin. An example of the composite CIP liner installed in a PVC pipe (Figure 188. the liner's total wall thickness of 14.0 mm) comprises of the following layers: PVC pipe, PP seam "tape" (the outer coated layer); thin layer of standard felt, glass fiber reinforced layer, standard felt "core layers", carbon fiber reinforced layer, and standard PP coated layer.

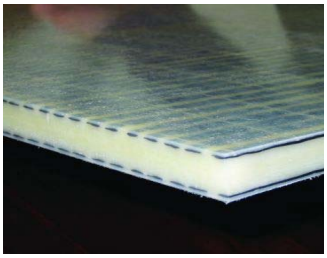


Figure 186. Sandwiched tube material used for one composite CIP (Shearer, 2007)

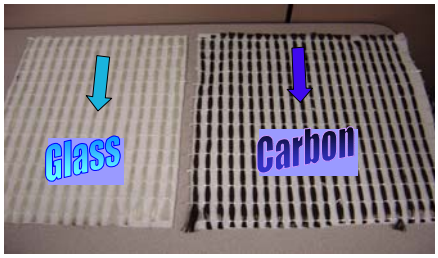


Figure 187. Carbon and glass fiber tows stitched onto thin layers of felt (Shearer, 2007)

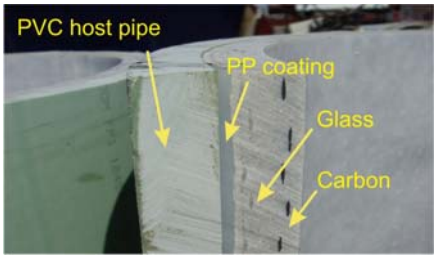


Figure 188. Cross-section of a composite CIP liner installed in a PVC host pipe (Shearer, 2007)

In another composite CIP liner, a non-porous inner membrane is sandwiched between two layers of structural fiberglass (Figure 189) and bonded to them. Felt fibers allow bonding of the inner membrane to the surrounding fiberglass layers with epoxy resin during installation. Beside this three-layered system, multilayered systems comprising up to four inner membranes between five layers of fiberglass can be made. Depending on the number of layers, the thickness of the pre-saturated liner ranges between 3.0 mm and 6.0 mm. With increased thickness, the physical properties improve considerably, e.g., flexural modulus per ASTM D790 increases from 600,000 psi to 1,000,000 psi (Poly-Triplex Technologies, 2009a). Third-party testing (SWL, 2004) in accordance with ASTM C497 D demonstrated a significant increase in culvert pipe strength after CIP relining with a composite CIP liner made with four layers of fiberglass, non-porous membranes in-between, and epoxy resin, compared to the original culvert pipe (Figure 190).

[1] *The term "tow" means a loose untwisted rope.*

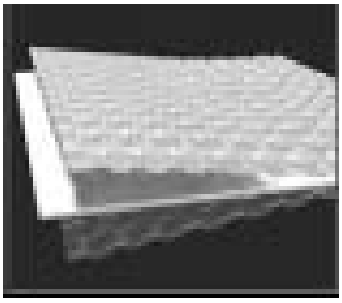


Figure 189. Three-layer system comprising a fiberglass layer, a non-porous felt/PVC membrane, and a fiberglass layer) (Robert Putnam, Poly-Triplex Technologies, personal communication)

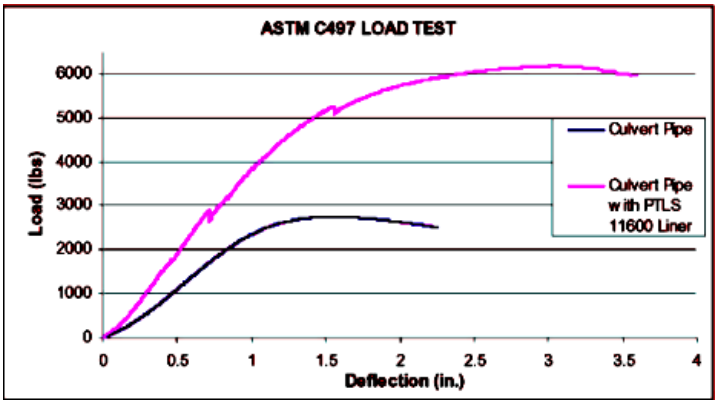


Figure 190. ASTM C497 load test of galvanized corrugated metal pipe before and after CIP relining (SWL, 2004)

3.1.3.3 Method Applicability

CIP liners can be installed in pipes of any material and shape (circular or non-circular), with or without corrugations on the inside surface of the pipe. The corrugations may be filled with cement grout prior to applying the liner.

Conventional CIPP is applicable in diameters between 6 in. and 108 in., and installation lengths range from 10 ft to 2,850 ft (Simonson and Peterie, 2002; McClanahan, 2007). Smaller diameters (e.g., 4 in. pipes) can also be CIP relined (WERF, 2006). Composite CIPP is used in pipes typically 48 in. in diameter or larger, although can be used in 36 in. pipes as well.

3.1.3.4 Construction Issues

3.1.3.4_1 Installation Procedure

Thornton et al. (2005) outlined the general installation procedure for conventional CIP relining:

- Test the atmosphere (in the insertion pits) for presence of toxic or flammable vapors, or the lack of oxygen
- Thoroughly clean the existing culvert using high-velocity jet cleaners
- Inspect the existing culvert (identify any protrusions, collapsed sections, deflected joints, etc)
- Clear line obstructions (excavate to remove and repair the obstructions if needed)
- Vacuum-impregnate the insertion tube with the specified resin under controlled conditions. Lubricate the tube before installation (apply lubricant to the fluid in the standpipe or directly to the tube).
- Setup flow bypass
- Invert the resin-impregnated tube (with hydrostatic head or air/steam pressure)
- Let the resin cure (while circulating hot water or steam throughout the liner with approved equipment; maintain the recommended pressures throughout the curing process)
- If heated water was used to cure the resin, drain the heated water from a small hole made in the downstream end and replace with the introduction of cool water into the inversion standpipe. Cool the liner to a temperature below 100°F before relieving the static head in the inversion standpipe. If air/steam was used to cure the resin, drain the air/steam through a small hole made in the downstream end and replace with the introduction of cool water into the guide chute. Cool the liner to a temperature below 113°F before relieving the pressure within the section.

- Cut and seal the termination ends with a resin mixture compatible with the installed liner if the liner does not fit tightly against the original pipe.
- Inspect the completed installation (CCTV or visually in man-entry pipes)
- Perform leakage or other testing to specifications
- Reconnect laterals and service connections with a robotic cutting device or manually where the diameter permits man-entry.
- Restore flow and initiate site cleanup.

Additional notes: A pre-installation video is typically made. The liner tube can be resin impregnated in the regional wet-out facility or on site.

Composite CIPP utilizes the same installation procedure (the liner is resin pre-impregnated and inserted by inversion or pull-in into the host pipe where it cures into the final product). However, one composite CIP liner is installed (Jerry Trevino, Protective Liner Systems, personal communication) by first manually applying resin directly onto the surface of the host pipe and then pressing the liner cloth into the resin (thus impregnated the cloth with the resin inside the host pipe; multiple layers can be applied), which is followed by the resin cure.

3.1.3.4_2 Liner Inversion or Pull-in

For inverting resin-impregnated liner tubes, air inversion is typically used for smaller diameters, i.e., 6 to 24 in. (Figure 191) and water inversion for diameters over 24 in. Water inversion can be carried out by either creating hydrostatic head with a tower (Figure 192) or applying simulated head through water pressure (Gearhart, 2008). Special inversion drums can also be used for inverting smaller diameters liners (WERF, 2006). Alternatively, the resin-impregnated liner tube can be pulled into the culvert using a winch (Figure 193). A calibration hose is then inverted into the center of the pulled-in tube using hydrostatic water pressure or pressurized air/steam.

Figure 191. Air pressure inversion (Jaques, 2008b)

Figure 192. Water inversion (Jaques, 2008b)

Figure 193. Liner being winched into the culvert (McClanahan, 2009)

3.1.3.4_3 Resin Cure and Cool Down

Ambient temperature cure can take a long time and the process can be accelerated by circulating hot water, air or steam through the pipe (Figure 194, Figure 195). In small diameters, once water reaches approximately 180ºF and interface temperatures are appropriate, curing takes approx 2 to 3 hours for water cure and 15 to 60 minutes for air cure. In large diameters, curing times can be 12 to 18 hours. Once cured, the liner is cooled to 100ºF. The pressure is then released and the ends are cut out (McClanahan, 2008).

UV-light resin cure utilizes a mobile light source that contains several UV lamps that operate at steady radiation intensity (wavelength range of 360 to 420 nm) and are optimally placed in the liner during the

curing process so that the liner is evenly lit even with varying channel diameters and line profiles. The curing process with UV light is very fast, e.g., curing speeds of 5 ft/min are possible (WERF, 2006)

Figure 194. Hoses for delivering hot water (McClanahan, 2009)

Figure 195. Steam cure (McClanahan, 2009)

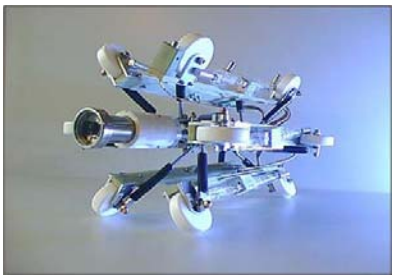


Figure 196. UV-light cure (McClanahan, 2009)

3.1.3.4_4 Set-in-Place CIP Liners

One composite CIP liner made of fiberglass cloth (E-type glass) and modified epoxy resin system (the resin has imbedded fibers for increased tear resistance of the liner) is installed as set-in-place rather than by inversion or winching. The application requires man entry into the pipe. With this system, mastic is applied first at approx thickness of 0.1 in. The fiberglass fabric is then cut into the required dimensions and pressed, using a putty knife, into the mastic to achieve full wetting of the fabric. With subsequent applications of the fabric, the edges are overlapped. Epoxy is applied between the overlapped edges to assure a monolithic construction. The fabric is top-coated with the mastic to insure complete saturation and encapsulation of the fabric. The finish lining systems has a minimum thickness of 0.125 in. The epoxy cures in 3 to 4 hours at 70°F to approx 75% of its strength (at this time the structure may return to service) and to its full strength in 4 to 5 days. Higher temperatures reduce the cure time and lower temperatures increase it (Jerry Trevino, Protective Liner Systems, personal communication).

3.1.3.5 QA/QC Considerations

Kampbell and Whittle (2003) reviewed the fundamentals of good QA/QC for CIP liners and provided guidelines for knowledgeable monitoring of the QA/QC results. CIP liners are manufactured in place and it is important that a sample of the finished wall section be obtained and analyzed for the specified finished thickness and material properties. Restrained samples, or flat plate samples, subjected to the in situ curing pressures and heating cycle are a must for assessing the structural properties of the finished liner. Sampling rates should be sufficiently frequent and line segments should be targeted for sampling based upon quality assurance needs and concerns. Line segments exhibiting early or incomplete curing, or resin slugs, or those at high risk of resin washout may deserve particular attention. Lines installed with significant deviations from the planned temperature and pressure schedules may also warrant testing. Visual inspection can help to identify discoloration, blisters, delamination, dry spots, etc. to target line segments for additional inspection and/or testing.

Kampbell (2005) reviewed the design parameters and the installation parameters necessary for long lasting cured in place pipe (CIPP).

NASTT (2006b) summarized QA/QC issues for CIPP liners saying that QA/QC should address both the component products (the resin and the tube) and its design and installation. The ability of the CIPP to withstand buckling was recognized as the key design consideration in CIPP design. ASTM standards for proper installation by insertion or pull in were listed.

3.1.3.6 Standards and Specifications

The following ASTM standards are associated with conventional CIPP relining:

ASTM D5813 covers specification, evaluation, and testing of materials used in the rehabilitation of existing pipes by the installation and cure of a resin-impregnated fabric liner. (Material standard)

ASTM F1216 and **ASTM F1743** describe the procedures for the reconnection of pipelines and conduits by the installation of a resin-impregnated, flexible tube which is inverted by use of a hydrostatic head or air pressure (F1216) or the pulled-in-place (F1743) into the existing conduit. (Installation standards)

ASTM F2019 is the third generic ASTM standard for CIPP liners (installation standard) that covers a tube composed of a fiberglass-reinforced material. It describes procedures for the reconstruction of pipelines and conduits 4 to 48 in. in diameter by pulled-in place installation followed by inflation with compressed air. The resin/fabric tube is cured by mixed air and steam.

Additional standards and specifications associated with CIP relining are listed in Thornton et al. (2005).

3.1.3.7 Example Case Histories

3.1.3.7_1 Conventional CIPP

Tingberg (2007) addressed site-specific challenges inherent in CIP relining of large diameter non-circular, concrete box culverts, and discusses design considerations, installation issues, and testing. The paper focuses on two case histories detailing the design through to installation.

Sukley and St. John (1994) reported on the use of CIP liners for the rehabilitation of corrugated metal culverts by the Pennsylvania DOT. The process (polyester fiber-felt and polyurethane resin) was installed in a 48-year old corrugated metal pipe on SR 0358 in Mercer County. Despite its relatively high cost, the project resulted in savings and the method was recommended for use statewide.

Indiana DOT installed CIP liners in multiple culverts in July 2003 and in December 2007 (Table 27) (Larry Vaughn, INDOT, personal communication).

Table 27. Selected culvert CIP lining projects by Indiana DOT

Location	Year	Culvert Diameter	Culvert Length
(SR 63 and US 41)	2003 (July)	18 in.	49 ft
		24 in.	48 ft
		24 in. × 36 in.	51 ft
INDOT (US 231 and SR 36)	2007 (Dec)	15 in.	48 ft
		24 in.	63 ft
		24 in.	53 ft
		15 in.	46 ft

Gwinnett County, GA completed a total of 136 culvert CIP lining projects up until April 2009 (Table 28). The majority of the lined pipes were of the corrugated metal classification, and the rest were clay, concrete and plastic pipes. Since April 2009, 12 CIP culvert lining projects were completed, all of which were cured using steam rather than hot water. Two corrugated metal culverts would be lined next, a 96 in. CMP 70 ft long in 2009, and a 102 in. CMP 234 ft long in 2010 (Frank Matticola, Gwinnett Co, personal communication).

Table 28. Culvert CIP lining projects in Gwinnett County, GA between Dec 2005 and Apr 2009

Item	Diameter	CIPP Liner Thickness	Cumulative Length
1	15 in.	7.5 mm	516 ft
2	18 in.	8.0 mm	5997 ft
3	21 in.	10.0 mm	2,021 ft
4	24 in.	10.0 mm	5,531 ft
5	30 in.	12.5 mm	5,098 ft
6	36 in.	16.0 mm	5,607 ft
7	42 in.	18.0 mm	3,332 ft
8	48 in.	21.0 mm	2,266 ft
9	54 in.	24.0 mm	1,649 ft
10	60 in.	28.5 mm	1,030 ft
11	66 in.	30.0 mm	223 ft
12	72 in.	32.5 mm	221 ft

City of Baytown, TX relined a 60 in. egg-shaped galvanized CSP culvert 90 ft long with a CIP liner (fiberglass and PVC vinyl liner tube, epoxy resin) in 2005. Relining was carried out in sections (Poly-Triplex of Texas, 2008.).

City of Brighton, CO relined a galvanized CSP culvert under the highway used for irrigation water (oval shape, 30 in. wide and 110 ft long) with CIP liner (fiberglass and PVC vinyl liner tube, epoxy resin) in 2002 (Dawn M. Hessheimer, City of Brighton, CO, personal communication).

3.1.3.7_2 Composite CIPP

Texas DOT relined a 660 ft long corrugated metal culvert, 48 in. in diameter, in Dallas with a composite CIP liner 2007. Total thickness of installed liner was 18.5 mm (Mike Bostic, TxDOT, personal communication).

Vermont Agency of Transportation (VTrans) relined a 145 ft long corrugated metal culvert, 66 in. in diameter, with a composite CIP liner in 2008. Total thickness of installed liner was 21.5 mm (Charlie Harding, VTrans, personal communication).

Florida DOT rehabilitated a 70 ft long corrugated metal culvert, 30 in. in diameter, under highway (US 98) in Destin, FL (Poly-Triplex Technologies, 2009b; Robert Putnam, Poly-Triplex, personal communication).

The City of Sylvester, GA, rehabilitated a 24 in.×72 in. double corrugated metal culvert under a busy highway. The liner was pulled in place and steam pressure cured. (Poly-Triplex Technologies, 2009c; Robert Putnam, Poly-Triplex, personal communication).

3.1.3.8 Advantages and Limitations

The main advantages of CIP relining are elimination of the need for excavation and grouting, and installation of a one piece (jointless) product that provides structural renewal with an expected 50-year service life. CIPP is a proven technology (it has been in use for 30 years), may be cost effective, and causes minimal traffic disruption. Small diameter installations can be completed in as little as one day.

The main limitations of this method are the need for flow bypass (unless the culvert pipe is empty at the time of rehabilitation), custom made tube is required for each installation, trained personnel are required, prolonged liner cure is needed for large diameters, there is potential for thermal pollution (if hot water was used to accelerate resin cure) and there is potential for adverse environmental impact (if styrene-based resins are used).

3.1.4. FOLD AND FORM / DEFORM AND REFORM

3.1.4.1 Overview

Two lining procedures involve the insertion of a thermoplastic pipe with an outside diameter slightly larger than the inside diameter of the host culvert pipe. The liner pipe is "folded" for easy winching into the host culvert pipe where it is subsequently expanded to closely fit the size and shape of the host pipe (Figure 197, Figure 198). These liners are categorized as either fold and form liners or deform-reform liners. While they are often referred to as "close-fit liners", this term is not used here since other lining processes produce close-fitting liners (e.g. CIP and helically wound liners).

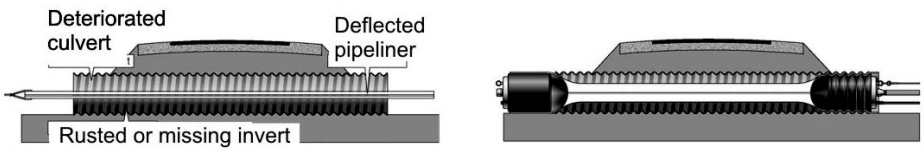


Figure 197. Two step installation. Left: A folded liner is pulled in. Right: A folded liner is expanded forming a tight-fitting new pipe (Roads and Bridges, 2001)

Figure 198. Cutout of a CMP rehabilitated with a fold-and-form liner (Roads and Bridges, 2001)

3.1.4.2 Materials Used

3.1.4.2_1 Fold and Form Liners

Fold and form liners are PVC-based products that are delivered to the site flattened (folded into a "U" or "C" shape prior to insertion into the host pipe) or folded into a "C" shape and coiled on large vertical drums. One manufacturer makes liners folded into an "H" shape for sizes 15 in. and larger (Figure 199). After being pulled into the culvert, the liner is re-rounded using steam and air pressure.

Figure 199. Common shapes of folded PVC liners: flat shape and "H" shape (Munson, 2008)

3.1.4.2_2 Deform-Reform Liners

Deform-reform liners are HDPE pipes that are manufactured in a round shape and cooled to set the memory to round, then reheated with warm water to a temperature lower than manufacturing and deformed into the "U" shape (the pipe is pushed through a former that folds it into a "U" shape or, as some call it, a "heart" shape, Figure 200), and this shape is temporarily held by a sleeve or plastic bands (Figure 201). Once the pipe is folded, it is coiled on large vertical drums and shipped to the job site (Figure 202). After being pulled inside the culvert, the liner is re-rounded by applying pressure to snap the bands that hold the deformed shape allowing the liner to revert back to its original shape. Either

pressurized steam (i.e., a combination of steam generated temperature and pressure) or hot water can be used.

Figure 200. An HDPE pipe deflected into a "heart" shape (Subterra, 2008)

Figure 201. Plastic bands hold a deflected pipe in shape (Long, 2005)

Figure 202. Deflected HDPE pipe with restraining bands, and coiled on a large vertical drum (Long, 2006)

3.1.4.3 Method Applicability

Whittle (2009) showed diameter range, DR range and max installation length for four fold-and-formed systems and one deform-reform system in the US (Table 29). Circular pipes ranging in diameter from 6 in. to 24 in. are routinely rehabilitated. Non-circular culvert shapes can also be rehabilitated, e.g. elliptical (see Example case histories). Lengths up to 1,500 ft can be relined.

Table 29. Technical envelope for four different fold-and-form liners on the US market (Whittle, 2009)

	PVC1	PVC2	PVC3	PVC4	HDPE
Diameter range	6 in.-12 in.	3 in.-24 in.	3 in.-15 in.	6 in.-16 in.	3 in.-24 in.
For "common" diameter: D DR range & typical DR Max length	 8 in. 32.5-26.0 (32.5) 1,500 ft	 8 in. 41.0-26.0 (35.0) 1,500 ft	 8 in. 41.0-32.5 (35.0 1,500 ft	 8 in. 34.0 (34.0) 500 ft	 8 in. 21.0-35.0 (32.5) 1,500 ft
For max diameter: D DR range & typical DR Max length	 12 in. 32.5-26.0 (32.5) 500 ft	 24 in. 50.0-60.0 (55.0) 650 ft	 15 in. 41.0-35.0 (35.0) 800 ft	 16 in. 34.0 (34.0) 250 ft	 24 in. 32.5-35.0 (35.0) Butt-fused

Applicability is not limited by culvert pipe type or condition unless the pipe has already collapsed (these liners can rehabilitate deteriorated pipes with ovality up to 10%, soil voids, and with offsets and bends). Deep pipes exceeding 30 ft have been rehabilitated. Both fold-and-form and deform-reform liners can be installed in corrugated culvert pipes, however, the installation can't be performed in live flow conditions.

3.1.4.4 Construction Issues

3.1.4.4_1 Installation Procedure

The following are steps that are normally required for both fold and form and deform-reform liners (modified from Thornton et al., 2005):

- Test the air in the pits for the presence of toxic or flammable vapors or the lack of oxygen.
- Clean the existing culvert structure using high-velocity jet cleaners.
- Inspect culverts for protrusions, sags, collapsed sections, etc, that may hinder liner installation and remove any obstructions found.
- Setup flow bypass.
- Heat the coil or reel containing the folded liner prior to insertion, if recommended by the

manufacturer.

- Insert a containment tube into the culvert by winching and inflate with low pressure and heat, if recommended by the manufacturer.
- Insert the deformed liner by winching.
- Expand the folded liner using heat and pressure. A rounding device may also be used in combination with heat and pressure. Maintain the temperature and pressure as required.
- Cool down the liner as required and relieve the expansion pressure.
- Trim down the terminating ends
- Inspect the completed installation (CCTV)
- Carry out leakage or other testing, as required
- Reconnect connections
- Restore flow if bypass was required and initiate site cleanup.

3.1.4.4_2 Fold and Form Liner Insertion, Forming and Cooling

Dayton (2008a, b) provided construction details for one fold-and-form relining system. The spool is placed into a hotbox where low pressure steam (280°F) from a boiler truck is used to heat the material to soften it until it is sufficiently pliable (i.e., as wet leather) to be pulled through the pipe (Figure 203). The liner is heated to a temperature recommended by the manufacturer, usually around 110°F, which takes about 1 hour. For winching, the "pulling nose" is made by folding the liner flat in half and drilling two opposing holes approx 12 in. from the edge, through which the chain is fed and hooked onto the cable. The liner is pulled through the pipe allowing it to protrude on both ends of the host pipe (Figure 204). A flow-through plug is inserted at one end of the liner and steam under low pressure is introduced into the liner. The liner material heats relieving stress induced from pulling, which is indicated by movement of the liner end with the plug (the opposite end of the liner is locked by the pulley system). The process continues until no liner movement can be observed. The stress relief process is then repeated at the opposite end of the liner.

Figure 203. Heating the liner with steam before insertion (Dayton, 2008a)

Figure 204. Liner protruding beyond the brick catch basin after being winched in (Dayton, 2008b)

Figure 205. Steam pressure being inserted (Dayton, 2007)

Dayton (2008a, b) also described steps that involve the PVC liner expansion and forming, and its cooling to ambient temperature. The liner is further heated using low-pressure steam until it is pliable enough for expansion. Heating time is determined by the length and SDR of the liner. At the end of the heating time, the process is switched from steam to compressed air. The pressure is increased to 18 psi thus inflating the liner to conform tightly against the host pipe. Once the liner is expanded, the pressure is reduced to 12 psi to hold it in place while an after-cooler blows in air at 80°F. The temperature at the exhaust end is monitored and when it drops to 100°F, the liner is hard enough not to collapse and the pressure is turned off. Depending on diameter and length, the liner cools in 1 to 2.5 hours. Water may be introduced into the compressed air during the cooling process to reduce the cooling time. The liner can be reheated, extracted and reinserted if any mistake has been made. After cooling, the liner is trimmed to

a minimum of 3 inches beyond the culvert for possible shrinkage during the process of cooling it to ambient temperature.

3.1.4.4_3 Deform-Reform Liner Insertion and Re-rounding

Deform-reform liners do not require pre-heating for installation (Long, 2006). The flexible liner is pulled off the coil and winched into the host pipe (Figure 206). Once in place, the ends of the pipe are closed off and a flow-through plug is inserted (Figure 207). Steam generated from a boiler truck is sent through the pipe causing the liner to expand and fit tightly to the inside of the host pipe (Griffin, 2007).

Figure 206. Insertion of a deformed liner from a drum (Griffin, 2007)

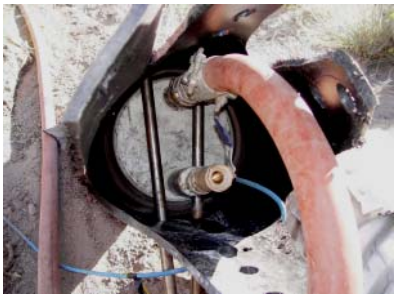


Figure 207. Pressurized steam expands and re-rounds the PE pipe (Long, 2006)

3.1.4.5 QA/QC Considerations

Whittle (2000) reviewed material characteristics that affect quality assurance, the window of field instability, as well as the structural capacity of pipeliner materials. The paper also outlined the engineering requirements that differ from traditional “direct burial” design. ASTM standards relevant for quality control procedures often used at manufacturing facilities were listed.

Kampbell and Whittle (2003) reviewed the fundamentals of good QA/QC for these liner technologies and provided guidelines for knowledgeable monitoring of the QA/QC results. As these liners are pre-manufactured products, quality control is simplified. All structural properties are established under ASTM prescribed QA/QC protocols common to all plastic pipe production and are confirmed on a specified sampling rate by batch, lot, or production run. The liners must merely be expanded to fit the ID of the host pipe. Preliminary wall thickness confirmation can be achieved prior to installation, and post-installation confirmation of the finished wall thickness (SDR) is easily and inexpensively accomplished (a restrained sampling technique per ASTM F 1871, F 1504, or F 1533, with laboratory testing). Post-installation QA/QC inspection is important to identify any stretch marks, thin spots, blow holes, waviness, lumpiness (not to be confused with conformance to host pipe anomalies), or discoloration from stretching, which could lead to further liner inspection and/or testing.

NASTT (2006e) summarized the QA/QC material verification, proper installation practices and post-installation procedures and practices, and listed ASTM standards that provide reference on accepted installation practices of thermoformed pipeliners.

3.1.4.6 Standards and Specifications

3.1.4.6_1 Fold and Form Liners

ASTM D1784 covers rigid PVC compounds and chlorinated PVC compounds for use in extruded or molded form like pipe and fitting applications. (Material standard)

ASTM F1871 and **ASTM F1504** cover the requirements and test methods for materials, dimensions, workmanship, flattening resistance, impact resistance, pipe stiffness, extrusion quality, and a form of marking for folded PVC pipe for existing sewer and conduit rehabilitation. (Product standards)

ASTM F1867 and **ASTM F 1947** cover the procedures for the rehabilitation of sewer lines and conduits by the insertion of a folded/formed PVC pipe that is heated, pressurized, and expanded to conform to the wall of the original conduit. (Installation standards)

Additional standards and specifications associated with fold and form relining are listed in Thornton et al. (2005).

3.1.4.6_2 Deform-Reform Liners

ASTM D3350 covers the identification of polyethylene plastic pipe and fitting materials (cell classification of materials). (Material standard)

ASTM F1533 covers requirements and test methods for deformed polyethylene (PE) liner for the rehabilitation of gravity flow and non-pressure pipelines. **(**Product standard**)**

ASTM F1606 covers the requirements for the installation of deformed polyethylene (PE) liner for pipeline rehabilitation. This practice applies to the rehabilitation of 3 to 18 in. diameter pipe in terms of installation (Installation standard)

Additional standards and specifications associated with deform-reform relining are listed in Thornton et al. (2005).

3.1.4.7 Example Case Histories

3.1.4.7_1 Fold and Form Liners

Georgia DOT (GDOT) used fold-and-form liners for the rehabilitation of corrugated metal culverts in 2001. Seven deteriorated culverts ranging from 15 in. to 30 in. in diameter and 40 to 80 ft in length were successfully relined. GDOT selected this method because it involved no digging, no chemicals were used, there was never need to block more than one lane of traffic for more than four hours, and it was the least expensive method identified by GDOT at that time (Roads and Bridges, 2001).

GDOT continued using this technology and by 2003 had rehabilitated 88 corrugated metal pipes in Georgia's District One. The average length was about 50 ft, although the length of some exceeded 200 ft, and the diameters varied between 15 in. and 24 in. The thickness of new liner varied between 0.35 in. and 0.55 in. The tensile strength of the finished liner was reported to be 4,100 psi and the modulus of elasticity rated at 155,000 psi (Public Works, 2003).

Kansas Turnpike Authority (KTA) used fold-and-form relining to repair two corrugated metal culverts under a roadway near Lawrence, KS in the fall of 2008. The culverts were 32 in. in diameter and approximately 70 ft long, showing extreme corrosion (one culvert had approx 40% of metal loss at the invert) and preliminary buckling failure (with Ed Patterson, KTA, personal communication).

City of Atlantic Beach, FL used fold-and-form relining to repair an 18 in.×29 in. elliptical CMP culvert, 50 ft long in the residential area that was on a verge of collapse. A 24 in. PVC liner 0.75 in. thick was used (Dayton, 2008b).

Jacksonville Naval Air Station, FL rehabilitated 12 RCP culverts (2,042 ft total length) that were between 6 in. and 30 in. in diameter using this method. A 0.5 in. thick PVC liner was used for culverts smaller than 15 in., and a 0.75 in. thick liner was used for 15 in. and larger diameters (Dayton, 2008a)

In Sarasota/Tampa area, a 24 in. HDPE corrugated storm sewer running under the school buildings was rehabilitated using fold-and-form liner in late 1990s. The HDPE pipe was cracking and splitting, allowing the surround soil to migrate into the pipe, which would create voids under the floor slab (Luke Whittle, Ultraliner INC, personal communication).

3.1.4.7_2 Deform-Reform Liners

Caltrans had a corrugated steel culvert under a highway near Big Sur, CA rehabilitated using deform-reform relining in 2007. The culvert was 18 in. in diameter and was badly deteriorated, with the bottom rusted completely in many places (Griffin, 2007).

Caltrans had approx 5,000 ft of culverts at various locations under Interstate 80 near Lake Tahoe, CA, and under Interstate 5 near Redding, CA, rehabilitated using deform-reform relining. The culverts ranged in diameter from 18 in. to 56 in. (Griffin, 2003).

Oregon DOT rehabilitated a 100 ft long 18 in. unreinforced concrete pipe (UCP) culvert using deform-reform relining in 2006 (George Vernon, Mill Creek Management Technologies, personal communication).

3.1.4.8 Advantages and Limitations

The main advantages of fold-and-form and deform-reform relining include the elimination of the need for excavation and grouting, and the one piece (jointless) final product with an expected minimum 50-year service life. Installation is straight-forward and fast, with minimal traffic disruption. The liner is manufactured in a controlled environment under stable conditions and the installation process does not change the physical properties of the liner material. No hazardous chemicals are used and no refrigeration is required during transportation of materials or for their storage. Reduction of cross area of culvert pipe is minimal, while flow capacity remains the same or is even improved due to the liner's smooth surface.

The main limitations of this method are diameter limitation (up to 30 in. diameter) and the need for flow bypassing (installations can't be performed in live flow conditions). Installation lengths are limited by pull-in forces or coil length (however, this is typically not an issue in culvert rehabilitation). Chemical grouting may be required at liner ends.

3.1.5. SEGMENTAL LINING (PANELS)

3.1.5.0 Synonyms

GRP, FRP and RTRP are synonyms abbreviations for fiberglass reinforced pipes (glass-fiber reinforced plastic; fiberglass reinforced plastic; and reinforced thermosetting resin plastic).

3.1.5.1 Overview

Segmental lining is used to rehabilitate man-entry pipes (medium to very large size) and is especially suitable for odd-shaped pipes. The method utilizes panels that are individually set in place inside the host pipe where they are bonded together, and the annular space is subsequently filled with a low viscosity, free flowing, rapid setting and high strength grout. The result is usually a structurally integrated composite pipe consisting of the existing pipe, grout and panels (Figure 208). However,

beside the composite design (Channeline International, 2009b), segmental lining is also suitable for stand alone or corrosion barrier designs.

The panels can be discrete 360° pipe segments of various shapes (i.e., short length, full-perimeter, and shaped as circular, oval, or box) or individual arc panels assembled into full-perimeter pipe either inside the pipe (Figure 209) or on the ground surface before insertion (Figure 210).

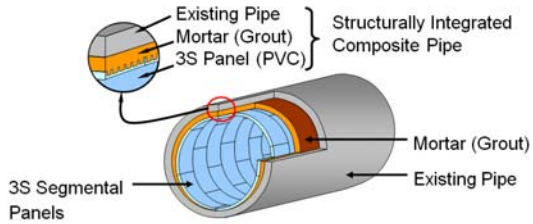


Figure 208. Segmental lining can create a structurally integrated composite pipe consisting of the existing pipe, high strength grout and panels (Kampbell, 2009)

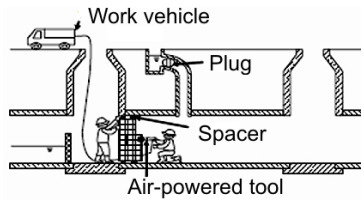


Figure 209. Assembly of panels inside the pipeline (Shonan Plastic, 2009)

Figure 210. Two-piece arc units being bonded together before insertion (Channeline International, 2009b)

3.1.5.2 Materials Used

GRP panels are composite materials. One example of sandwiched GRP panel construction is shown in Figure 211, where the panel is made of several layers. (1) The inner structure has a 1.5 mm resin-impregnated coating (isophthalic or vinyl ester resin) that acts as a corrosion barrier, and an inner skin made of several layers of resin impregnated multi-axial engineered fabric, e.g., CSM[2] impregnated with unsaturated polyester resin. (2) A central core is made of silica and resin that are mixed and evenly applied to a specific thickness. (3) An outer skin is made of several additional layers of multi-axial fabric and CSM-resin. The outer surface is treated with a bonded graded aggregate to enhance adhesion to the annular grout, which is used during the installation phase (Channeline International, 2009b).

PVC panels are made of mold-injected plastic (Figure 213) identical to the material of PVC wastewater pipelines.

Corrugated metal structural plates are made of field bolted galvanized steel plates (Figure 213) or aluminum, in a variety of shapes, including round, pipe-arch, arch, ellipse (horizontal and vertical), and underpass shapes. Complete structures can be pre-assembled on-site and lifted into place without heavy or specialized equipment.

High strength structural grout is a mix of ordinary Portland cement, fly ash and water, which when correctly mixed and installed develops compressive strength in the range of 3,000 to 5,000 psi at 28 days (Channeline International, 2009b).

[2] *Chopped Strand Mat (CSM) is a form of reinforcement used in GRP that consists of glass-fibers laid randomly across each other and held together by a binder. The binder dissolves in resin, and the material easily conforms to different shapes when wetted out.*

Figure 211. Cross-section of sandwich panel construction (Channeline International, 2009b)

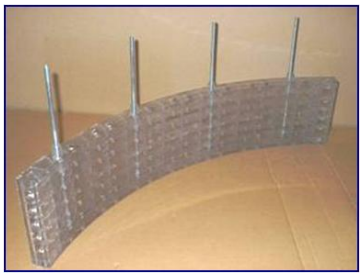


Figure 212. Transparent PVC panels (Kampbell, 2009)

Figure 213. Corrugated steel structural plate (CONTECH, 2009c)

3.1.5.3 Applicability

There is no theoretical limit to the shape and size of pipe that can be rehabilitated (with multi-piece segmental construction). The most commonly encountered pipe shapes are: circular, oval, egg shaped, elliptical, flattened elliptical, arch barrel, box shaped, and flattened box shaped (Channeline International, 2009b).

PVC panels are used in circular pipes in. 40 to 157 in. in diameter and box culverts ranging in size from 40 in.×40 in. to 197 in.× 197 in., as well as in other pipe shapes (Kampbell, 2009).

3.1.5.4 Construction Issues

3.1.5.4_1 Placing the Panels inside the Culvert Pipe

If GRP panels are used, access pits are dug at suitable locations along the length of the culvert structure (if needed) and the crown of the culvert is removed to allow insertion of sections. For short runs, excavations at each end of the culvert are typically adequate. Multi-segmented panels are bonded on site to “full perimeter” liner segments using epoxy bonding compound. The segments are lowered into the pipeline opening using a suitably rated crane until they rest in the invert of the culvert at the pit location. A special hydraulic trolley (Figure 214) is used to transport each liner segment along the length of the host pipe to the required location. Once in position, the liner segment is centralized and chocked using hardwood wedges. Each liner segment is connected to the previously installed one by means of the socket and spigot joint. Once butted together, the joints are injected with a flexible mastic epoxy adhesive/filler. The annular space between the liner and the host pipe is filled with a low viscosity, free flowing, rapid setting, and high-strength grout (Channeline International, 2009b)

If lightweight PVC panels are used, they are carried by hand into the existing pipe where they are bolted into rings (Figure 215, Figure 216) using air-powered tools. For centering the liner inside the culvert pipe, spacers of appropriate size are positioned approximately 3 ft apart (Shonan Plastic, 2009).

Figure 214. GRP segment on installation trolley (Channeline International, 2009b)

Figure 215. Paneling inside a pipe (Kampbell, 2009)

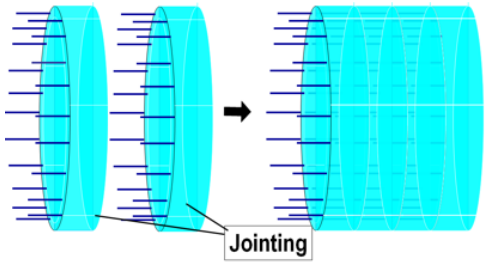


Figure 216. Ring-paneling with PVC panels (Kampbell, 2009)

3.1.5.4_2 Grouting

In GRP liners, the grout ports are drilled at intervals along the crown of the liner and the grout pumped into the annular space (Figure 217) (Channeline International, 2009b).

In PVC liners, injection holes are drilled at locations predetermined according to the pipe diameters, at distances of approximately 15 ft, and the grout is pumped through them (Figure 218). The panels are made semi-translucent to allow for positive confirmation of the grouting of the annulus (Figure 219) (Shonan Plastic, 2009).

Figure 217. Injection of grout through grout ports into the annular space (Channeline International, 2009b).

Figure 218. Grout pumping into the annular space (Kampbell, 2009)

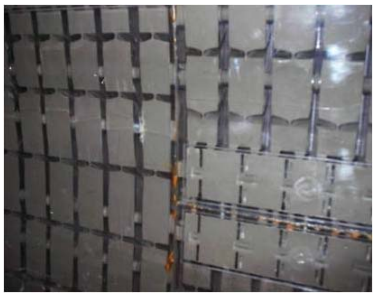


Figure 219. Grout visible through semi-translucent PVC panels (Kampbell, 2009)

3.1.5.4_3 Buoyancy and Flotation

The practicalities of buoyancy and flotation can be dealt with in several ways, but the most common is to undertake the grout filling of the annulus in three or more stages, dependent upon the height and diameter of the liner (Channeline International, 2009b).

3.1.5.5 QA/QC Considerations

NASTT (2006f) outlined QA/QC issues for thermoset panels. QA/QC should address the component products (the resin, the fillers and the reinforcing agents), the design (thickness, host pipe configuration, corrosion resistance, hoop strength and fit), and installation (joint fit, installation method, lateral restoration and grouting).

For QA/QC of GRP panels, daily and batch testing of each production run should be carried out by the QC department to verify conformity with design dimensions (wall thickness, ID, OD, height and width), bending and flexural modulus, tensile tests, socket and spigot fit, Barcol hardness, and visual appearance (Kevin David, Channeline International Ltd, personal communication).

3.1.5.6 Standards and Specifications

ASTM D3262 covers machine-made glass-fiber-reinforced thermosetting-resin (fiberglass) pipes (Material standard).

ISO/TR 10465-1 describes the procedures for underground installation of flexible GRP pipes (Installation standard).

ASTM A761 covers corrugated steel structural plate, zinc-coated, used in the construction of pipe, pipe-arches, arches, underpasses, and special shapes for field assembly.

ASTM A796 covers the structural design of corrugated steel pipe and pipe-arches, ribbed and composite ribbed steel pipe, ribbed pipe with metallic-coated inserts, closed rib steel pipe, composite corrugated steel pipe, and steel structural plate pipe, pipe-arches, and underpasses for use as storm sewers and sanitary sewers, and other buried applications.

AASHTO M 167 covers corrugated steel structural plate, zinc-coated, used in the construction of pipe, pipe-arches, arches, underpasses, and special shapes for field assembly. Appropriate fasteners and accessory materials are also described. The pipe, arches, and other shapes are generally used for drainage purposes, pedestrian and vehicular underpasses, and utility tunnels.

3.1.5.7 Advantages and Limitations

The main advantages of segmental lining are the ability to rehabilitate very large and odd-shaped pipes, and to accommodate alignment changes and short radius bends. The method can be effective for localized repair. Lightweight panels (PVC) are easy for handling with less labor and equipment requirements.

The main limitations of this method are the diameter limits (sufficient for man-entry) and the need for excavating access pits (with GRP panels).

3.1.5.8 Example Case Histories

3.1.5.8_1 PVC Panels

In Orlando, FL, see-through PVC panels were used to rehabilitate a 72 in. concrete storm sewer pipe in 2005. Two pipe segments were rehabilitated, which were 130 ft and 97 ft long (Griffin, 2009; Ray Pavlic, National Liner, LLC, personal communication).

3.1.5.8_2 GRP Panels

Doherty et al. (2005) reported three projects in which GRP segmental lining was used to rehabilitate storm or combined sewer pipes in Toronto, ON, Canada:

- A 2,343 ft long box shaped poured-in-place concrete storm sewer was relined in 2003. The sewer consisted of four sections whose inside dimensions varied considerably: 40 in.×60 in. (942 ft), 36 in.×48 in. (459 ft), 48 in.×43 in. (384 ft), and 51 in.×51 in. (558 ft).
- A circular corrugated steel storm sewer, 66 in. in diameter and 1,178 ft long, was relined in 2003 using 5 ft long panels with ID 49 in. and OD 52 in.;
- A 1,736 ft long combined sewer was rehabilitated in 2004/05. The sewer consisted of several sections having three different shapes: an arch top brick sewer 48 in.×60 in. (1,440 ft), egg-shaped brick sewer 47 in.×62 in. (108 ft) and 52 in. circular iron pipe (1,735 ft). GRP panels ranged in thickness from 20 mm to 35 mm.

In Buffalo, NY, a 140 ft long arched culvert, 12 ft × 6 ft, was relined in 2003 using a two piece GRP segmental liner (Channeline International, 2009a)

3.1.5.8_3 Metals Panels

In Scranton, PA, a stone-arch culvert (U.S. Route 11/Pittston Ave) was rehabilitated after a massive sinkhole developed leading to the collapse of a large section of U.S. Route 11. Aluminum structural plate and galvanized steel tunnel liner plate arch, 13 ft span x 11 ft 4 in. rise, were used (CONTECH, 2009b).

Connecticut DOT rehabilitated a 335 ft long, twin 14 ft ellipse shaped steel structural plate pipe structure (installed in 1964) by relining it with a galvanized steel tunnel liner plate and aluminum tunnel liner plate (Project #80-125). On the upstream end, the steel plate was installed with high strength mortar lining for hydraulic efficiency. On the downstream end, the aluminum plate was installed without lining to reduce water velocity at the outlet. New structures have diameters from 154 in. to 172 in. (CONTECH, 2009b).

Oregon DOT rehabilitated a 160 ft long 48 in. CMP culvert under Hwy 26 in 2007 using CMP plate seals (George Vernon, Mill Creek Management Technologies, personal communication).

3.1.6. GROUT-IN-PLACE LINERS

3.1.6.1 Overview

Grout-in-place liners (GIPL) are made of thermoplastic liner tubes (PVC or HDPE) that are installed with anchors (e.g., V-shaped studs) on the outside of the liner, which serve as a spacer to the inner surface of the host culvert pipe thus creating annular space that is filled with high-strength cementitious grout. The grout is the primary structural element.

3.1.6.2 Materials Used

The liner tube can be a studded HDPE liner (Figure 220) inserted by inversion or winching in, PVC strip installed by spiral winding (Figure 221), or soft panels (PVC, HDPE, etc) installed by slipforming (Figure 222).

Figure 220. Studded HDPE liner (Whittle, 2007)

Figure 221. Winding a PVC strip (Whittle, 2008)

Figure 222. Slipforming with PVC sheets (Whittle, 2008)

Paterson (2000) stated that studded HDPE liners can be installed as three different systems:

- The basic system contains a single HDPE liner. The height of studs determines the thickness of annular space, which is typically between 0.4 in. and 0.75 in.

- A preliner system that also includes a smooth HDPE liner, which is usually required in areas near or below the groundwater level to ensure dilution-free grouting.
- A double system that incorporates another HDPE liner to create a larger void space around the existing culvert pipe wall that will be filled with structural grout, thus increasing the structural capacity of the lining system. Double-liner systems are used in applications requiring additional structural load-bearing capacity and high-security containment.

Grouts used with grout-in-place liners have adequate mechanical locking to the host structure and the liner tube, low shrinkage and bleeding, little segregation in water and high compressive strength, e.g., 2,800 psi after 7 days of curing and 4,900 psi after 21 days with PVC GIPL (McAlpine, 2009a). Compressive strength of grouts used with HDPE GIPL is higher (Whittle, 2008) and can exceed 13,000 psi after 28 days (Cooper, 2000).

Grout-in-place HDPE liners may have a “self-cleaning invert” feature due to a special floor texturing that causes micro turbulences: during periods of increased flow in the pipe, any deposits are swept-up from the liner surface, directed to the flow centre and transported downstream (Whittle, 2008; Trolining GmbH, 2008).

3.1.6.3 Applicability

Grout-in-place liners can rehabilitate circular pipes from 6 in. to 120 in. or larger, however the method is applicable to any geometry.

The method can be applied in any culvert pipe material and shape. Corrugations on the inside surface do not hinder the method applicability. Pipe condition is generally not a limitation, e.g., the pipe can be corroded, deformed, and near collapse.

Spiral winding machines with a custom form frame can rehabilitate both circular and non-circular pipelines and are typically used in medium to very large diameter pipelines, e.g. 36 in. to 12 ft ×15 ft and larger. The pipeline can have curved alignment with radius of curvature at least 7 times pipe diameter (Tackenberg, 2007).

Installation lengths can go up to 625 ft (Sterling et al, 2009).

3.1.6.4 Construction Issues

3.1.6.4_1 Host Pipe Preparation

The following are the steps involved in preparing the host pipe (Whittle, 2008): inner dimensions of host pipe are measured, inner surface condition inspected, the pipe is cleaned, and rebar repaired as necessary.

3.1.6.4_2 Installation of HDPE Grout-In-Place Liners

Cooper (2000) provided details of the installation procedure. These liners and preliners are factory manufactured based on measured dimensions of the host pipe. The sheets are formed into cylinders and welded along their axial length to create a double-weld seam. If a preliner is used, it is folded and winched in first, and re-rounded to conform to the walls of the host pipe using a 5 psi air pressure. A studded liner is winched in place (Figure 223) inside the preliner and similarly re-rounded. Once both liners are in place, the open ends of all liners are sealed together using extrusion welding. After the annulus is sealed, the inflatable plugs are placed at the ends and the interior filled with water and pressurized. The grout is mixed to consistent water/cement ratio and viscosity within a specified range,

and injected/poured under controlled head into the annulus between the liner and preliner through grouting ports (Figure 224). The injector is sampled throughout the injection process for proper water/cement ratio and viscosity, and evidence of expansion/swelling. After the grout injection has been completed, the hydrostatic pressure inside the liner is maintained while the grout cures to its nominal strength. The installed liners are inspected with CCTV. The translucent nature of some HDPE liners allows easy inspection.

Figure 223. Winching in a studded HDPE liner PVC (Whittle, 2007)

Figure 224. Grouting and air release ports installed after the annular space between the HDPE liner and preliner has been sealed (Whittle, 2007)

3.1.6.4_3 Installation of Spirally Wound PVC Liners

3.1.6.4_3a Installation Using a Winding Machine

Tackenberg (2006, 2007) outlined the basic installation process using a winding machine:

- Install winding machine
- Wind the liner
- Remove winding machine and install internal bracing
- Inject grout
- Remove bracing

In the pre-installation phase, inner dimensions of the host pipe are measured, inner surface condition inspected, any obstructions and projecting points mitigated, and the pipe cleaned. Hydroblasting is recommended to remove buildup of grease and other foreign matter from the walls as well as all loose tiles and aggregate. Inverts need to be in good condition or repaired if necessary for the winding machine to run smoothly. Although the installation can be done in live flow conditions, a project with unpredictable flow levels may require a flow diversion.

The winding machine is assembled on the ground and inserted into the pipe (Figure 225). It has a custom form frame designed for the interior dimensions of pipe and a roller system that travels around the frame (Figure 226). Hydraulic motors are used for pulling in the winding profile (strip) and locking it into place. After winding, a bracing form (Figure 227) is installed as a support for the grout and to maintain clearance between the host pipe and the liner. The support jacks prevent the profile from floating or collapsing during the injection of the grout. Typically one support jack is needed every 7 ft, so this step is the most time consuming step during the installation. Bulkheads are constructed at the upstream and downstream sections and the grout is injected.

Figure 225. Winding machine with a custom form frame (Tackenberg, 2007)

Figure 226. A close view at the winding machine (Tackenberg, 2006)

Figure 227. A bracing form (installed and centered) before the grouting (Tackenberg, 2006)

3.1.6.4_3b Manual Installation

Spirally wound grout-in-place PVC liners can also be manually installed in man-entry pipes. McAlpine and Anderson (2005) discussed cleaning, lining and grouting of large diameter pipe. The PVC liner is delivered on site in coils. The liner has "T" shaped ribs on one side (the ribs provide a mechanical anchor for the PVC liner as the annular gap would be filled with suitable grouts) and smooth surface on another (to form the flow surface). The liner is taken into the pipe's interior by simply pulling it from the inside of the bound coil. One end of the liner, usually at an upstream starting point, is formed into a circular hoop of desired diameter. The edge joints of adjacent windings are joined together by a second "joiner" strip that is inserted with an air hammer (Figure 228). The joiner strip has a co-extruded rubber gasket that forms a compression seal making the joint watertight.

Figure 228. Hammering a profile locking strip in place (Danby Pipe Renovation, 2007)

Figure 229. Rebar beam bolsters bolted to CMP (Danby Pipe Renovation, 2007)

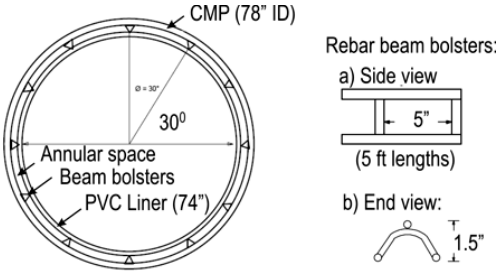


Figure 230. Rebar beam bolsters designed in a CMP (Danby Pipe Renovation, 2007, modified)

Prior to strip winding, steel welded wire mesh can be placed inside the pipe or rebar beam bolsters can be installed to serve as annulus spacers (Figure 229 and Figure 230). The annular space is subsequently filled in with grout. Because the PVC liner is a relatively low stiffness formwork, the grout must be placed in lifts of limited vertical rise. Drilling holes in the liner at the desired height can control the lift heights.

3.1.6.4_4 Lining with PVC Panels

In man-entry non-circular pipes, ribbed plastic panels can be used for lining. The panels can be made to match any shape of the host culvert pipe. The panels are typically 12 in. wide and can be made to match specific job requirements (the length is practically limited by the ability of trucks to deliver them on site, e.g., up to 50 ft). The panels are placed inside a pipe and locked together on the edges (snapped) to form a continuous liner.

Figure 231. Flat and “corner” PVC panels installed in the invert and in sharp corners of a low-rise arch culvert (Danby Pipe Renovation, 2007)

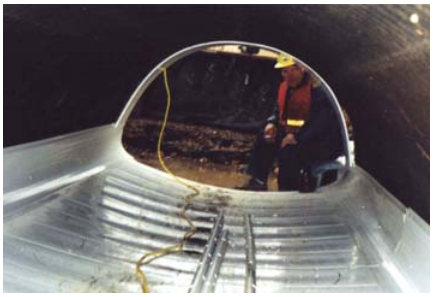


Figure 232. Arch PVC panels complete the slipforming (Danby Pipe Renovation, 2007)

3.1.6.5 QA/QC Considerations

Whittle (2008) stated that quality controls for grout-in-place relining must ensure consistent wall thickness, consistent design modulus (elastic modulus), and consistent corrosion resistance (chemical composition).

Key quality assurance checks for GIP HDPE liners in the field include water tightness of all joints and seams, monitoring of grout properties at ambient temperatures, and monitoring humidity changes. Key quality control tests in the field are viscosity, water-to-cement ratio, weight, temperature, and changes in volume (expansion or contraction during curing). Prism specimens should also be poured for post-construction strength tests in a laboratory (Grant Whittle, Ultraliner Inc, personal communication).

3.1.6.6 Standards and Specifications

3.1.6.6_1 PVC Liners

ASTM D1784 covers rigid PVC and chlorinated PVC compounds for use in extruded or molded forms like pipe and fitting applications (Material standard).

ASTM F1697 covers requirements and test methods for materials, dimensions, workmanship, stiffness factor, extrusion quality, and a format for marking for extruded PVC profile strips used for field fabrication of PVC liners used in host pipes ranging in diameters from 6 in. to 180 in. and for similar sizes of non-circular pipelines such as arched or oval shapes and rectangular shapes (Product standard).

ASTM F1735 covers the requirements and test methods for materials, dimensions, workmanship, extrusion quality, and form of marking for extruded PVC profile strips used for field fabrication of PVC liners for existing man-entry (36 in. to 144 in.) sewer and conduit rehabilitation (Product standard).

ASTM F1698 describes the procedures for the rehabilitation of sewer lines and conduits of man-entry sizes (36 in. to 144 in. in vertical dimension) by the installation of a field-fabricated PVC liner. After installation of the liner, cementitious grout is injected into the annular space between the liner and the existing sewer or conduit. Rehabilitation by this installation practice results in a rigid composite structure (PVC/grout/existing pipe) (Installation standard).

ASTM F1741 describes the procedures for the rehabilitation of sewer lines and conduits 6 in. to 180 in. in diameter by spiral winding of an extruded thermoplastic strip into the existing pipeline. The procedures using stationary installation equipment and the traveling installation equipment are covered (Installation standard).

3.1.6.6_2 HDPE Liners

North American standards that will cover these liners are in development, but have not yet achieved consensus peer review. The following are applicable European standards:

- DWA-M 143-10 (Sewer rehabilitation with studded liners)
- ATV M127, Part 2 (Structural design)
- DIN EN 1610 (Tightness test)
- DIN EN 295 (Abrasion resistance)
- DIN EN 196 (Grout properties)
- DVS 2225 (Liner manufacture by hot wedge welding method)
- ISO 1133 (Physical properties of HDPE)
- ISO 527 (Mechanical properties of HDPE)
- DVGW W 270 (Suitability for drinking water application)

3.1.6.7 Example Case Histories

In Stone Mountain, GA, several corrugated metal culverts were rehabilitated using an HDPE grout-in-place liners: a 36 in. culvert 114.6 ft long, a 48 in. culvert 71 ft long, a 48 in. culvert 66.4 ft long, and a 60in. culvert 73.1 ft long, in 2004 and 2005 (Bob Cowhig, Stone Mountain Memorial Association, GA, personal communication).

In Tuscaloosa, AL, a twin corrugated metal culvert 144 in. x 78 in. (elliptical) and 80 ft long was rehabilitated using an HDPE grout-in-place relining in 2008 (Chad Christian, City of Tuscaloosa, personal communication).

South Carolina DOT reported using spirally wound grout-in-place liners to reline a corroded culvert under highway I-95 in Santee, SC. The culvert was circular, 84 in. in diameter and 212 ft long. The project took about three weeks to complete and was performed smoothly with minimal interruption of highway traffic (only one highway line had to be closed at the time, for a total of 12 hours) (Thomas Breland, Jr, SCDOT, personal communication).

In Covington, GA, a 78 in. CMP culvert was rehabilitated with a 74 in. ID grout-in-place liner. The PVC strip was spirally wound manually over rebar beam bolsters that were 1.5 in. high (Danby Pipe Renovation, 2007).

Two additional examples of manual spiral winding for grout-in-place liners in culvert pipes include the rehabilitation of two CMP culverts (72 in. and 84 in.) for Oregon DOT (Danby Pipe Renovation, 2007).

In Boise, ID, a low rise arch CMP culvert, 44 in. x 72 in., was rehabilitated by lining with PVC panels and subsequent grouting, thus forming a grout-in-place liner (Danby Pipe Renovation, 2007).

3.1.6.8 Advantages and limitations

The main advantages of grout-in-place relining are the ability to rehabilitate culverts with practically any size and shape, ability to accommodate bends, the fact that no excavation is required, this system results in a full structural rehabilitation, and the method is often cost competitive with other methods especially in large diameters and non-round geometries (lower mobilization costs and shipping costs as no refrigeration of materials is required).

The main limitation is the cost of structural grout if the liner is designed with considerable thickness. The required thickness can be reduced if the liner is designed as a composite structure with the host pipe, but this requires good surface cleaning of the existing culvert pipe, e.g., high pressure water jetting, and the use of mechanical anchors, especially in CMP culverts. Reduction in flow area depends on the

liner thickness and can be significant. However, reduction in hydraulic capacity due to reduced cross-sectional area is typically at least partially mitigated by an improved Manning's *n* value. Also, the method can't be applied with flow in the pipe and therefore a bypass may be needed.

3.1.7. SPRAYED-ON LINERS

3.1.7.1 Cementitious Liners (Shotcrete)

3.1.7.1_1 Overview

Shotcreting is the method in which cement mortar or concrete is pneumatically applied (jetted at high velocity) onto the surface of the structure being repaired. Shotcreting can be classified as a wet-mix process (all the ingredients including the water are mixed prior to introduction into the delivery equipment) or a dry-mix process, also known as gunite (all ingredients except the water are mixed and fed into the delivery equipment, and the water is added at the nozzle) as shown in Figure 233 (Hilton, 1990). The choice between gunite and the wet-mix process depends on various factors, e.g., project size (quantity of shotcrete required), application rate required (cubic yards per hour or shift), dust (could be an issue in a confined space and less is produced by wet-mix shotcrete), rebound (wet-mix shotcrete generally generates less material bouncing off the shooting surface), and required pumping length (Sulman, 2009).

Wire mesh reinforced cement mortar lining involves application of two layers of cement-mortar that are separated by a wire mesh (Figure 235). The first layer of cement-mortar (0.5 in. thick) is applied to the inner wall of the host pipe in the conventional manner (i.e., using gunite). A wire mesh is placed against the first coat of lining using overlapping joints, and the second layer of cement-mortar (0.5 in. thick) is applied by trowel over the wire mesh (AWWA, 2001).

Figure 233. Gunite application (REED, 2009)

Figure 234. Gunite lining inside the pipe (Queensland Gunite, 2009)

Figure 235. Wire mesh installed in the pipe; guniting will follow the mesh level and trowelling will complete the liner above it (Queensland Gunite, 2009)

Cement mortar lining can also be installed by spincasting whereby the equipment traversing through the pipe (Figure 236) applies a continuous thin layer of cement mortar (thickness typically between 0.25 in. and 0.5 in., and up to 2 in. if multiple layers are used) onto the interior of a deteriorated culvert pipe (Figure 237). A high-speed rotating applicator device is used to provide a densely compacted liner of uniform thickness and thorough coverage of the pipe interior surface. In the same pass through the pipe, the sprayed layer can be trowelled by either rotating spatulas fitted to the spraying machine or a simple tubular shield pulled behind the spraying machine (ISTT, 1998). However, no additional trowelling or surfacing is necessary with some centrifugal casting systems (AP/M Permaform, 2009). In small diameter pipes, the equipment is remotely operated. For large diameter pipes, an operator of the machinery can enter the pipe and control the flow of mortar, speed of travel, and the speed of rotation for the lining machine head. An example of a large diameter lining machine is shown in Figure 238.

Figure 236. High-speed rotating applicator device for centrifugal casting of cement mortar inside the culvert (Henning, 2009)

Figure 237. A thin layer of cement mortar applied by spincasting onto the cleaned concrete culvert pipe (Henning, 2009)

Figure 238. Large diameter cement mortar lining machine (Proline Pipeline Protection, 2008)

3.1.7.1_2 Materials Used (Shotcrete Mortars and Concretes)

Morgan et al. (1989) reviewed conventional and specialty shotcretes (steel fiber reinforced, polypropylene fiber reinforced, and wire mesh reinforced). ACI (1991) presented state of the art on fiber reinforced shotcrete. US Army Corps of Engineers (1993) provided guidance on the shotcrete selection and proportioning.

Conventional shotcrete concretes are made of Portland cement, aggregates and chemical admixtures. Gunite mixture is made from 1 part Portland cement and 4 parts sand (REED, 2009). Special concretes often employ admixtures (e.g., polymer modifiers) that inhibit corrosion and enhance chemical resistance. For producing a freeze-thaw durable shotcrete, aggregates susceptible to frost attack should not be used and the maximum size of the aggregate should be lowered to reduce susceptibility to freeze-thaw deterioration (Hilton, 1990).

US Army Corps of Engineers (1993) outlined benefits of adding fibers to the shotcrete mixture, i.e., the added ductility of the material (unreinforced shotcrete is a brittle material that experiences cracking and displacement when subjected to tensile stresses or strains), as well as added energy absorption capacity and impact resistance. Fibers used in shotcrete include steel fibers, glass fibers, and synthetic fibers.

Steel fiber reinforced shotcrete (SFRS) is a mortar or concrete containing discontinuous discrete steel fibers, which are pneumatically projected at high velocity onto a surface (Indian Concrete Journal, 2003). Typical fiber lengths for shotcrete range between 0.75 in. and 1.5 in., and the fibers are used in the amount of 2% by volume of shotcrete (US Army Corps of Engineers, 1993). Banthia et al. (1992) assessed the influence of fiber geometry in steel fiber reinforced dry-mix shotcrete on the rebound characteristics and hardened shotcrete properties. Properly designed, SFRS can reduce or even eliminate cracking, a common cause for concern in plain shotcrete (Indian Concrete Journal, 2003).

Glass fiber reinforced shotcrete (GFRS) is made by adding glass fibers to the cementitious mix to improve flexural, tensile and impact strength. Alkaline resistant glass fibers are used. For glass fiber reinforced concrete (GRFC) spray-up, the optimum fiber length is 1.5 in. to 2 in. The properties and proportional mixes are discussed in PCI (2001).

Girard (2006) provided an introduction to GFRS. The material is often used to make large, lightweight panels (considered lightweight because of the thinness of the material; GFRC concrete weighs on average about the same as ordinary concrete on a volume basis). When spray-applied, the fluid concrete mixture (minus fibers) is sprayed out of a gun-like nozzle that also chops and sprays a separate stream of long fibers. The concrete and fibers mix when they hit the form surface. Glass fiber is fed off of a spool in a continuous thread into the gun, where blades cut it just before it is sprayed. Chopped fiber lengths tend to be much longer (about 1.5 in.) than fibers that get mixed in, since long fibers would ball up if they were mixed into the concrete before spraying. Typically spray-up is applied in two layers. The first

layer is the face coat, much like a gel-coat in fiberglass. This face coat usually has no fibers in it and is thin, often only about 0.125 in. thick. The second, or backer layer, has the fiber in it. The action of spraying on the fibers results in random orientation, much like the layers in plywood. Spray-up permits very high fiber loading using very long fiber length. GFRC made using the spray-up method yields the greatest strength. However, the equipment required to do spray-up is expensive, often costing more than $20,000.

Polypropylene fiber-reinforced shotcrete (FRS) is made by adding polypropylene fibers to the cementitious mix to improve strain capacity, toughness, impact resistance, and crack control. Collated fibrillated polypropylene (CFP) fibers with fiber lengths between 0.5 in. and 2.5 in. are used, having the primary benefit of control over thermal and drying shrinkage cracking when added in the amount of 1 to 2 lbs of fibers per cubic yard of shotcrete and increased shotcrete toughness when added up to 10 lbs of fibers per cubic yard (US Army Corps of Engineers, 1993). Loevlie (2008) stated that this kind of mortar develops a high compressive strength (6,000 psi in 7 days), and that is more acid and abrasion resistant and more impermeable than ordinary concrete. Fiber-reinforced shotcrete (FRS) proportional mixes are discussed in Morgan et al. (1989).

3.1.7.1_3 Method Reviews

Hilton (1990) prepared a brief state-of-the-art review of shotcrete covering material properties, applications, finishing, curing, durability, and failures in shotcrete installations. Most shotcrete failures are found to be related to poor preparation of the substrate surface, and the general consensus is that good quality shotcrete is a durable material even when used in severe environmental situations.

Walker and Guan (1997) found that cement mortar lining systems were very economical but provided limited corrosion protection in a corrosive environment. Furthermore, they concluded that cement mortar did not bond to the steel surface but was instead held in place by its rigidity and shape.

New York DOT recommends lining with shotcrete if the structural integrity of culvert is sufficient and corrosion on the entire circumference of culvert is minor, generally less than 20% total perforations. If signs of minor corrosion are limited to the bottom of culvert (i.e. less than 30% of the bottom one-fourth of the pipe is perforated), paving the invert with Portland cement concrete (PCC) is the preferred method (NYDOT, 2001).

3.1.7.1_4 Applicability

The method is applicable in circular pipes ranging in diameter from 12 in. to 140 in. Reinforced cement mortar lining is only applicable in many-entry pipes.

Maximum installation length is about 650 ft in robotic applications and about 50 to 60 ft in man-entry applications for practical reasons (this also depends on safety regulations). Corrugations in the pipe do not hinder use of shotcrete, but the pipe must be completely empty (no live flow conditions) and cleaned (Caltrans, 2003).

3.1.7.1_5 Example Case Histories

Loevlie (2008) reported several case histories of relining culverts using fiber-reinforced shotcrete (FRS) such as:

- In Milwaukee, WI, approx 9,000 ft of pipe with 60 in. ID was relined applying a 0.5 in. thick shotcrete lining. The individual runs were up to 500 ft long (Kristian Loevlie, Shotcrete Technologies, Inc, personal communication)
- In Grand Junction, Mesa County, CO, a 36 in. diameter culvert 110 ft long was relined in less than two hours

- In Elbert County, CO, two corrugated metal culverts, 96 in. in diameter and 85 ft long, were relined in one day (Loevlie, 2008).

In Oldman River Dam in Alberta, Canada, a drainage tunnel, 10 ft in diameter, was relined using fiber-reinforced shotcrete (FRS) in 1987 (Morgan, 2000a).

3.1.7.1_6 Advantages and Limitations

The main advantages of cement mortar lining are the ability to provide protection against corrosion and abrasion, restore flow capacity, and provide structural repair if sufficient material thickness is used. No excavation is required.

The main limitations include the relatively long setting time and the relatively slow strength gain of the installed liner. The culvert must be completely free of water and flow bypass may be required. Extensive surface preparation is needed in most cases.

3.1.7.2 Polymer Based Coatings and Liners

3.1.7.2_1 Overview

Polymer based coatings and liners involve the application of a layer of polymer material by either spincasting (Figure 239) or manual spraying using spray guns (Figure 240). ASCE (2010) defines coatings as applications where only a corrosion barrier is created, and liners as applications where a corrosion barrier and/or a structural repair are provided.

Figure 239. The spincast system uses centrifugal force to apply the material to pipe walls over lengths up to 700 ft (RLS Solutions, 2009)

Figure 240. A spray gun is used for applying the material to pipe walls in man-entry pipes (Warren, 2007)

3.1.7.2_2 Materials Used

3.1.7.2_2a Epoxy

Although spray-on epoxy is mostly used for rehabilitation of potable water pipes, it can also be effectively used to line culverts (Thornton et al, 2005). Epoxy can be applied as protective coatings against corrosion and for eliminating infiltration/exfiltration.

Epoxy coatings are typically100% solids and solvent-free (i.e., they do not require a solvent to keep the binder and filler parts in a liquid suspension form). There are several advantages of such coatings over conventional liquid coatings: they emit zero or near zero volatile organic compounds; they can produce much thicker coatings than conventional liquid coatings without running or sagging; they produce less hazardous waste than conventional liquid coatings; generally they have fewer appearance differences between horizontally coated surfaces and vertically coated surfaces than liquid coated items (Walker and Guan, 1997).

Fiber reinforced polymer composites (FRPCs) contain high performance fibers embedded in a polymer matrix (Figure 242). The matrix serves to provide continuity to the composite, distribute applied loads among fibers, support the slender fibers against buckling, and protect the fibers from physical and environmental damage. FRPC materials have high strength-to-weight ratios, are generally resistant to corrosion, and are lightweight and thus relatively easy to apply (Warren, 2002)

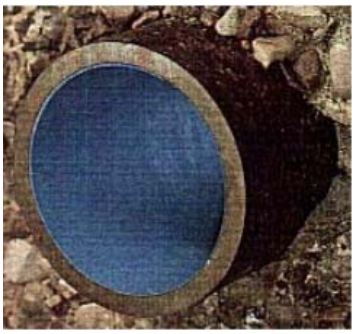


Figure 241. Epoxy-lined pipe (Proline Pipeline Protection, 2008)

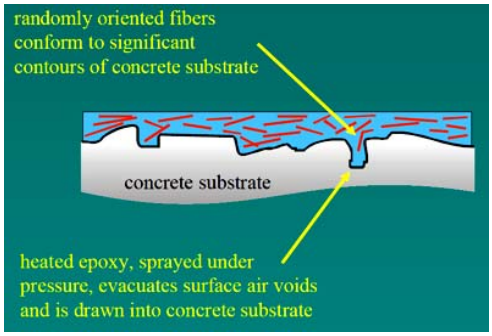


Figure 242. Spraying of fiber reinforced polymer composites (Warren, 2002)

3.1.7.2_2b Polyurethane

Sprayable polyurethanes are polyol based products blended with isocyanate. When mixed in a 2:1 ratio (2 parts of “B” polyol to 1 part of “A” isocyanate, proportioned by weight), a rigid coating is produced (permitting up to 4% elongation by ASTM D638) that provides both structural enhancement and corrosion protection. A 1:1 mixing ratio produces a flexible coating (permitting 43% elongation as assessed by ASTM D638) that offers only corrosion protection (Jerry Gordon, Sprayroq, personal communication).

Guan (2003a, 2003b) reviewed the chemistry, history and developments of 100% solids elastomeric polyurethane and 100% solids rigid polyurethane. Most pipe rehabilitation field applications have been traditionally based on 100% solids elastomeric polyurethane, but since the mid 1990s, the trend in North America has been towards the development and use of 100% solids rigid polyurethane coatings.

3.1.7.2_2c Polyurea

Polyurea coatings and liners are based on isocyanates/amines (Primeaux, 1989). They combine application characteristics such as rapid cure, even at temperatures well below 0°C, and insensitivity to humidity, with physical properties such as high hardness, flexibility, tear strength, tensile strength, chemical and water resistance. The resulting surface offers good weathering and abrasion resistance. The systems are 100 percent-solids, making them compliant with the strictest VOC regulations (Broekaert, 2002).

For structural enhancement, the liner is sprayed “high build” (the term typically implies thickness roughly between 0.25 and 1 in.) as the liner is designed to resist soil loads, traffic loads and hydrostatic groundwater pressure. Like cement mortar lining, polyurea lining of this thickness, holds in place as a result of its rigidity and shape, and the adhesion (i.e. surface preparation) is not as critical as with protective coatings (Donald Dancey, Innovative Painting & Waterproofing Inc, personal communication).

Figure 243. High build polyurea lining in pipe (Joseph, 2009)

Allouche and Steward (2009) evaluated structural enhancement of corrugated metal pipes using spray-on polyurea. The testing of four different polyurea formulations sprayed at thickness between 0.2 and 1 in. overall confirmed (1) extremely short return-to-service time of the pipe, (2) small ecological footprint during installation, (3) significant film thickness can be constructed although multiple passes might be needed, and (4) cost savings compared to other methods such as CIPP.

3.1.7.2_3 Applicability

The technology is applicable in all pipe shapes and types (steel, concrete, PVC, cast and ductile iron, asbestos cement, wood, corrugated metal pipes) but the pipes must be completely empty of water, dry and clean (Stephane Joseph, Acuro, personal communication). Spincasting is applicable in small diameter circular pipes, typically in a diameter range between 3 in. and 36 in., and up to 700 ft in distance (RLS Solutions, 2009). In man-entry pipes, the method is applicable in any pipe size and shape, and the installation length is typically limited to 450 ft (Thornton et al, 2005). Safety regulations can also limit the installation length.

3.1.7.2_4 Installation

3.1.7.2_4a Epoxy

Epoxy liners are predominantly installed in man-entry pipes with manual spraying.

One system on the market (two components and 100% solids) is installed with a plural component spray application system, which pre-heats the product, mechanically ratios the two components, mixes and delivers the homogeneously blended product to the spray gun (airless or air-assisted). Application thickness is between 0.06 in. and 0.25 in. per application layer. For quality assurance, at least two coats are recommended. When applying multiple coats, no more than 18 hours at 70°F should be permitted to pass between coats. Recoating is usually performed between 2 and 18 hours after the previous coat. Initial set generally occurs within 6 hours at 70°F. Curing continues for 7 to 14 days (Jim Henke, RLS Solutions, personal communication).

Another system is also using a plural component spray-on system for spraying the material. The epoxy component utilizes a 2 parts base to 1 part activator mix ratio by volume. No thinners are utilized. The coating is applied in thickness up to 0.750 in., and multiple coats can be applied to a max thickness of 1 in. The cure time is about 2 hours at 77°F. Additional coats are applied within one hour (Jane Warren, Warren Environmental Inc, personal communication).

3.1.7.2_4b Polyurethane

This material is sprayed onto the prepared surface through an airless spray gun using appropriate ratio system for mixing the components (see 3.1.7.2_2b). Application thickness is up to 1 in. or greater. The material begins to gel in about 8 seconds, with a tack free condition after one minute. Within 30 to 60

minutes, the initial cure is completed and the structure is capable of accepting flow. The complete curing continues for the next 4 to 6 hours (Jerry Gordon, Sprayroq, personal communication).

Surface preparation is essential for successful application. The surface must be clean (free of oil, grease, rust) and dry (polyurethanes react to water instantly having bubbling and blistering reaction) (Baron**,** 2007).

3.1.7.2_4c Polyurea

Polyurea is applied in thickness between 0.020 in. and 1 in., though maximal thickness is theoretically unlimited (Stephane Joseph, Acuro, personal communication).

The product cures rapidly with 5 to 8 seconds gel time, 12 to 15 seconds tack free time, and 24 hours return to full service. Application thicknesses from 0.125 to 1 in. can easily be achieved (for a high-built liner). The product can be sprayed directly onto concrete, metal, wood or brick substrates. If substrate is uneven or slightly damp, a primer is recommended (e.g., water blown high density foam that fills voids and creates an even surface) (Donald Dancey, *Innovative Painting & Waterproofing Inc*, personal communication).

3.1.7.2_5 Example Case Histories

3.1.7.2_5a Epoxy

Several case histories of applying fiber reinforced polymer composites (FRPCs) for spray-on lining of tunnels, sewer pipes, manholes, and aqueducts are listed on a web site (Warren Environmental, 2009) although no case history of culvert rehabilitation is included among them.

3.1.7.2_5b Polyurethane

In Norristown, PA, a galvanized corrugated steel culvert pipe, 60 in. in diameter and 1,800 ft long, was spray relined with polyurethane (100% solids elastomeric) in 2007. The lining was sprayed in one coating at thickness 0.3 in. with a proprietary heated plural component spray system. The application required very clean and very dry surface. The applied surface preparation procedure is proprietary for the system used (Jerry Gordon, *Sprayroq Inc*, personal communication)

3.1.7.2_5c Polyurea

Several agencies have already approved polyurea coatings for rehabilitation of culverts and sewer pipelines, e.g., Virginia DOT, Florida DOT, Ohio DOT, whereas polyurea coatings are under review in 35 other states for DOT evaluation nearing final approval stage (Hunting, 2008).

In 2008, City of Vaudreuil-Dorion, near Montreal, Canada, used a high-built polyurea to rehabilitate over 5,000 ft of cast and ductile iron water mains. The liner was applied in thickness of 0.12 and 0.2 in., in pipe diameters of 6 and 10 in. respectively (Joseph, 2009).

In Springerville, AZ, Tucson Electric Power (TEP) rehabilitated a water pipe intake into the power plant applying a polyurea lining in 2009. The pipe consisted of 1,800 ft of 96 in. and 620 ft of 72 in. diameter steel pipe. The existing mortar lining was first demolished and hydro/abrasive blasting was performed on the pipe interior surface to prepare it for the polyurea lining. The lining was applied in thickness of 0.06 in. using a combination of robotic plural-component spray equipment and hand-spray methods in areas inaccessible to robotic systems (Allouche and Steward, 2009).

In Chicago, IL, a partially deteriorated concrete storm line was experiencing large infiltration (caused by the acidic degradation of the joint seals and several radial cracks in the elliptical pipe that occurred due to the loss of soil stabilization) causing soil loss and voids around the pipe. The pipe was rehabilitated in 2000 by spray-applying a 0.75 in. thick semi-structural polyurea lining (Inspar Robotic Technologies, 2005).

Virginia Department of Transportation (VDOT) rehabilitated multiple deteriorated corrugated metal highway culverts by spray-applying polyurea lining in 2005 and 2006. The culverts diameters were ranging from 12 in. to 96 in. and the liner thicknesses from 0.50 in. to 1.35 in. Prior to lining, the culverts were prepared with high pressure water blasting. (Inspar Robotic Technologies, 2007).

One case study of polyurea spray-on rehabilitation of PVC pipes is the rehabilitation of a damaged PVC power cable conduit in Tampa, FL. The conduit was damaged while a concrete duct bank was installed under the existing highway (in the process of concrete pouring). The conduit was 8 in. in diameter and was rehabilitated with 0.15 in. thick polyurea lining that was installed robotically using a rotational grouting apparatus (Inspar Robotic Technologies, 2006).

3.1.7.2_6 Advantages and Limitations

The main advantage of polymer-based coatings and liners is the ability to provide protection against corrosion. Some also provide structural enhancement. No excavation is required.

The main limitation is that the culvert must be completely free of water and flow bypass may be required. An extensive surface preparation is essential for successful application with some systems.

3.1.7.3 QA/QC Considerations for Sprayed-on Liners

NASTT (2006a) outlined QA/QC issues for coatings and lining (cementitious, epoxies, urethanes and ureas). Referenced were material standards, most important installations issues (surface preparation, variables such as weather, safety, confined spaces, product viscosity, moisture tolerance, etc), and measures to ensure proper application (training of qualified applicators and proper equipment). The importance of testing and inspection of applied coatings and linings was recognized.

Muenchmeyer (2004) discussed elements of a QA/QC program that can minimize or prevent coating

failures (Table 30), and outlined key specification issues and customer acceptance criteria. Good verifiable quality controls and testing documentation during construction were identified as critical components for the long term success of corrosion protection coatings. The paper also discussed warranties and the importance of regular project inspections.

Table 30: QA/QC for polymer coating (based on Muenchmeyer, 2004)

Quality assurance plan	Quality control plan
▪ Complete description of the project site and structure condition specifying precautions, if applicable ▪ Defined QA criteria ▪ Defined QC verifications during construction ▪ Proposed safety plan for the work execution ▪ Schedule for product sampling ▪ Required submittals and certifications for the project and on the products to be used ▪ Training certifications for the Applicator	▪ Written verification that all QA requirement have been met ▪ Documentation that all safety requirements have been implemented ▪ Verification of coating materials submittals, delivery and use ▪ Pre-construction inspection of surfaces documenting their condition ▪ Inspection of equipment for surface preparation and coating application (applicability, operational condition, and manufacturer's approval) ▪ Documentation of environmental and service condition (temperature, humidity, pH, flow, infiltration, etc) ▪ Inspection and verification of surface preparation ▪ Measurements of film thickness of applied coating ▪ Inspection of film continuity (visual and holiday testing) ▪ Adhesion testing ▪ Inspection and documentation of post-inspection repair procedures ▪ Re-testing requirements of areas found to be deficient and repaired ▪ Other test requirements recorded and verified by the inspector

Muenchmeyer (2005) evaluated a failure of coatings installed in one rehabilitation project in 1996, and described lessons learnt for future applications. The project included the use of epoxy coating in approximately 50 large diameter manholes (the conclusions apply to coatings in general), which failed soon after the 5-year warranty expired for the following principal reasons: 1) the coating was installed with non-uniform thickness that varied between 0.06 in. and 0.30 in. (thinner non-monolitic areas failed), and 2) the exposed aggregate was in many areas left un-coated and the pinholes were not repaired thus leaving underlying concrete to corrode at more rapid rate. Prior to the 5-year warranty expiration, the coatings were inspected by non-experienced inspectors who did not identify any problems. Among the lessons learnt are: 1) experienced inspectors and regular annual inspections are essential (extended warranty has no value otherwise), and 2) good field advice should be considered (the applicator indeed recommended a cementitious coat prior to epoxy coating that the owner disregarded).

3.1.7.4 Standards and Specifications

Morgan (2000b) listed shotcrete guides and specifications.

AWWA C602 covers cement-mortar lining of pipelines from 4 in. to 144 in. in diameter.

ACI 506.2 provides a useful basis (but only limited guidance) for the preparation of detailed specifications for a variety of different shotcrete constructions

ACI 506R provides detailed information on materials and properties of both dry-mix and wet-mix shotcrete. Most facets of the shotcrete process are covered, including application procedures, equipment requirements, and responsibilities of the shotcrete crew.

European Specification for Sprayed Concrete (EFNARC, 1996a) deals with concrete or mortar which is pneumatically placed onto a surface. Both wet and dry processes are covered. The appendix covers the admixtures for sprayed concrete: definitions, specifications, requirements, reference concrete mixes and test methods. EFNARC (1996b) covers the execution of spraying of concrete or mortar. EFNARC (1999) provide a commentary on the Specification by giving an explanation of the requirements.

3.1.8. PIPE BURSTING

3.1.8.1 Overview

Pipe bursting is a construction method of trenchless pipe replacement in which deteriorated culvert pipes are replaced with new pipes of the same or somewhat larger diameter. The bursting tool is passed through the pipe breaking it into fragments if the pipe is brittle, or slicing through it if the pipe is ductile (also known as pipe splitting), and the new pipe is simultaneously pulled in (Figure 244). Most pipe bursting projects are performed using the pneumatic or static pull systems. Figure 245 shows entry into a culvert under a roadway that will be replaced by bursting.

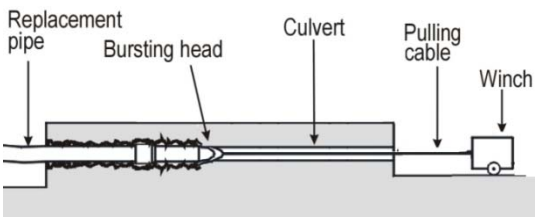


Figure 244. Pipe bursting of culverts (Simicevic and Sterling, 2001)

Figure 245. Entry into the culvert to be burst (Tric Tools, 2008)

3.1.8.2 Materials Installed

The typical replacement pipe installed by pipe bursting is an HDPE pipe since these chemically inert, they can readily flex, they maintain their circular shape, and have memory to return to its initial shape when kinked. Service life can also extend over 100 years. Other pipe types installed using pipe bursting include fusible PVC pipe, retrained joint PVC pipe, ductile iron pipe, and vitrified clay pipes.

The ASCE Manual of Practice for Pipe Bursting Projects (Najafi, 2007) reviews different types of replacement pipes providing advantages and limitations for each type of pipe.

Matthews and Allouche (2009) investigated a new segmented PVC pipe for trenchless pipe rehabilitation or replacement that is produced in 3 ft long sections and has bell-and-spigot ends but no profile socket (i.e., flash joint).

3.1.8.3 Applicability

Pipe bursting can replace circular pipes up to 54 in. diameter. The length is typically limited to 750 ft (Sterling et al, 2009).

Applicability is not limited by culvert pipe type (although corrugated metal pipes present a challenge as describes in 3.1.8.4_3) or condition. Replacement can be performed in live flow conditions.
Most favorable bursting projects involve pipes that were originally installed by trenching or open cut because the fill material surrounding them is usually conducive to pipe bursting. Upsizing depends on the soil conditions as well.

3.1.8.4 Construction Issues

3.1.8.4_1 Installation Procedure

Pipe bursting of culverts is typically performed in the following steps (modified from Simicevic and Sterling, 2001):

- Excavate entrance and reception pits as needed
- Connect the replacement pipe segments into a continuous pipe, if needed
- Set up (a) winch and pulling cable, or (b) hydraulic pulling unit and pulling cable, or (3) rigid pulling rods (if static pull)
- Install air supply hose through the replacement pipe and attach to the bursting head (if pneumatic bursting)
- Install a lubrication hose, if applicable
- Attach a pulling cable (or rod) to the bursting head
- Pipe burst and simultaneously pull in the replacement pipe
- Remove the bursting head
- Remove the hoses
- Perform leakage or other testing, if required
- As necessary, complete the project by constructing or modifying head- and wing-walls on the ends of the culvert
- Site restoration

3.1.8.4_2 Pit Excavation

Pipe bursting in general requires insertion and exit pits (Figure 244, Figure 246, and Figure 247), unless it is performed in areas where the culvert enters the side of the hill or embankment and is accessible without excavating pits.

Figure 246. Bursting ready to start (Eddie Ward, TT Technologies, personal communication).

Figure 247. The pull starts with ram, pulley wheel and resistance plate in place (Tric Tools, 2008)

3.1.8.4_3 Bursting of Corrugated Pipes

Bursting of corrugated metal pipes is challenging because corrugated metal tends to bunch up during a standard bursting application. A specially designed tool (Figure 248) can be used that has a cutting sleeve designed to keep the pipe rounded: two large blades shear and separate the metal pipe and prevent ovaling. The expander at the rear of this tool separates the sheared pipe and also pulls in the new replacement pipe (HDPE) simultaneously. A portion of CMP pipe might be extracted out of the ground instead of being sliced open and left in place (Figure 249).

Another technology uses a bigger than standard blade (Figure 250) to cut through the corrugated metal pipe. The impact from the pneumatic tool along with the blade design prevent the corrugated metal pipe from compressing.

Figure 248. Bursting head with cutting sleeve and rear expander (Eddie Ward, TT Technologies, personal communication).

Figure 249. The last 8 ft long section of CMP pipe extracted from the ground (Eddie Ward, TT Technologies, personal communication).

Figure 250. Bursting head with bigger than standard blade (Tric Tools, 2008)

3.1.8.4_4 Lubrication

Bentonite lubrication is used in various bursting situations to reduce friction. Friction also increases with the depth of the host pipe. Some soils, like beach sand, will not remain in the expanded state long enough for the installation of new product pipe. Bentonite lubrication is used in these situations to help maintain the annular space created as the tool travels through the host pipe (Orton, 2007).

3.1.8.5 QA/QC Considerations

Pipe Bursting Best Practices (NASTT, 2005) itemized QA/QC for butt fusion, a quality assurance plan for pipe bursting, quality controls during pipe bursting, and QA/QC testing and verification. Also listed were specification items that are the owner's responsibility and those that are the contractor's responsibility, as well as submittal requirements.

NASTT (2006d) outlined that QA/QC may be ensured through laboratory testing of materials at the owner's or third party laboratory (materials verification). In the field, QA/QC of installation practices includes measuring heave/settlement on the surface, potential vibration from the bursting operation, monitoring strain in the new HDPE pipe, and subsurface monitoring of existing utilities. Pressure testing or CCTV are the final step of QA/QC. Knowledgeable and trained inspectors are important for QA/QC in pipe bursting projects.

3.1.8.6 Standards and Specifications

ASTM C1208 covers manufacturing, quality assurance testing, inspection, installation, field acceptance testing, and product marking of vitrified clay pipe to be used in microtunneling, pilot tube, sliplining, or pipe bursting.

ASTM D 3212 covers joints for plastic pipe systems intended for drain and gravity sewage pipe at internal or external pressure less than 25 ft head using flexible watertight elastomeric seals. Test requirements, test methods, and acceptable materials are specified.

ASTM D3350 covers the identification of polyethylene plastic pipe and fitting materials (cell classification of materials).

ASTM D 2657 describes the general procedures for making joints with polyolefin pipe and fittings by means of heat fusion joining techniques.

PPI (2006a) specified the generic butt fusion joining procedure for field joining of polyethylene pipe. This standard has been adopted by most pipe manufacturers and is also referred to in ASTM D2657.

ANSI/NSF 14 covers physical and performance requirements for plastic piping components and related materials.

Additional standards related to pipe bursting are listed in Simicevic and Sterling (2001) and in Najafi (2007).

3.1.8.7 Example Case Histories

Dekalb County, GA, had a 15 in. corrugated steel pipe culvert 100 ft long replaced and upsized with a 24 in. HDPE pipe in 2005. A 14 in. bursting tool with 24 in. rear expander was used. The original CMP culvert remained in the ground, after shear opening and expansion, except for the last 8 ft long section that was pushed out (Eddie Ward, TT Technologies, personal communication).

In San Francisco, CA, a corrugated steel culvert under the roadway, 10 in. in diameter and 85 ft long was replaced with 10 in. fused HDPE pipe in October 2008. The pull took 40 minutes and the whole project was completed in one day with minimal excavation and disturbance (only a pedestrian detour was needed). (Tric Tools, 2008)

King County used pipe bursting in Black Diamond, WA in 2000 to replace and upsize a 700 ft long 30 in. VCP culvert with 42 in. HDPE pipe (George Vernon, Mill Creek Management Technologies, personal communication).

3.1.8.8 Advantages and Limitations

The main advantages of pipe bursting include the installation of a new pipe, ability for pipe upsizing and reduction of necessary excavation by 85% or more compared to open cut replacement. (If there is access from the end, then no excavation is needed.) The method is a proven technology with 60,000,000 ft installed worldwide (Nicholson, 2008). It is often more cost effective than open trenching in urban environments.

The main limitations of this method are inapplicability for already collapsed pipes or difficulties that arise when existing pipe composed of brittle material has had point repairs with ductile material. Pipe bursting can cause ground heave or settlement above or at some distance from the culvert, especially in dense sand, when the culvert pipe is shallow and ground displacements are primarily directed upward, and when significant diameter upsizing is performed. In addition, pipe bursting is not applicable when the host pipe experience significant sagging and deviation from the original grade.

3.1.9. OTHER IN-LINE REPLACEMENT METHODS

3.1.9.1 Overview

Other in-line culvert replacement methods include pipe ramming, tunneling and pipe jacking.

Pipe ramming uses a pneumatic tool to drive a pipe or casing into the ground while consuming the existing culvert pipe. ASCE manual (Najafi, 2008) outlines the process in detail.

Pipe jacking installs a new pipe segment by segment in place of an existing culvert pipe using hydraulic jacks that are located in a jacking pit (Figure 251). The method can be used as long as the shield and the jacking pipe can consume the existing culvert and the new pipe allows man-entry.

Tunneling utilizes a simultaneous advancement of a protective shield and the manual removal of the existing pipe in pieces. Replacement is carried out with liner plate, i.e., liner plate rings, typically 16 in.

long, are installed instead of a jacking pipe (Figure 252). Hydraulic cylinders located in the tunnel shield push against the most recently assembled liner plate ring advancing the shield forward. As the shield advances itself, it creates the space for adding a new ring (a jacking unit is not required, nor a jacking pit). After the liner plate is in place, it can be sliplined with a new concrete pipe or a wire mesh reinforced shotcrete lining can be applied.

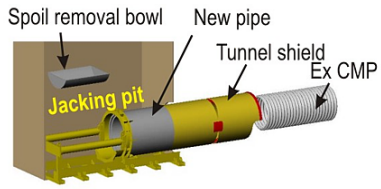


Figure 251. Replacement with jacking pipe (Tenbusch and Tenbusch, 2008)

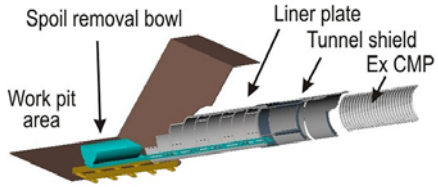


Figure 252. Replacement with liner plate (Tenbusch and Tenbusch, 2008)

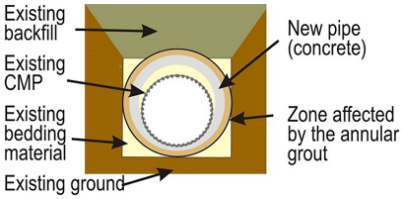


Figure 253. Placement of a new RCP through the existing CMP culvert (Tenbusch et al., 2009)

During the process of pipe replacement by tunneling, some of the surrounding soil is removed along with the existing pipe to accommodate the outside diameter of the new pipe (most of the soil removed consists of the original bedding, as shown in Figure 253). The grout and bentonite lubricant permeate the granular bedding material that remains, stabilizing the soil envelope around the new pipe and potentially limiting groundwater migration along the new pipe. If pipe upsizing is involved, the face material is excavated as needed (Tenbusch and Tenbusch, 2008).

3.1.9.2 Materials Installed

Jacking pipe materials available for this method include vitrified clay, polymer concrete, reinforced concrete and steel pipes.

Liner plates are made of steel that can be as thick as 0.375 in. The plates can be galvanized and coated with different coatings such as tar epoxy.

3.1.9.3 Applicability

The method can replace pipes of any type (corrugated metal, concrete) or shape (circular and non-circular). The method is suitable for pipe upsizing. The installed pipe must be large enough to allow man-entry (36 in. and up). The length of installation is not limited.

3.1.9.4 Construction Issues

3.1.9.4_1 Installation Procedure

Steps normally required for the replacement with jacking pipe (Al Tenbusch, Tenbusch Inc, personal communication):

- Inspect the culvert (identify any connecting pipes, protrusions, roots, sediment), however, man-entry should not be allowed in a culvert that has begun to fail.
- Excavate the pit
- Setup flow bypass if necessary (if there is water in the culvert and the pit preventing construction to take place)
- Position the jacking unit inside the pit
- Attach a protective shield to the first segment of the replacement pipe.

- Place the assembly into the pit.
- Prepare bentonite for lubrication.
- Install the first pipe segment by pushing. Remove the pieces of host culvert pipe as necessary
- Continue installing the remaining pipe sections.
- Inspect the installed pipe (man-entry visual inspection)
- Perform leakage or other testing, if required
- Grout the annular space.
- Restore flow (remove bypass pumping) if applicable.
- Remove the equipment and complete the site restoration and cleanup.
- As necessary, complete the project by constructing or modifying head- and wing-walls on the ends of the culvert.

Steps normally required for replacement with liner plate (Al Tenbusch, Tenbusch Inc, personal communication):

- Inspect the culvert (identify any connecting pipes, protrusions, roots, sediment), however, man-entry should not be allowed in a culvert that has begun to fail.
- Setup flow bypass if necessary (if the construction proceeds during the rain event)
- Set up the backstop (in areas where the culvert enters the side of the hill or embankment, a heavy piece of construction equipment is suitable to push against)
- Install the first liner plate segment. Remove the pieces of host culvert pipe as it fractures.
- Continue installing the remaining pipe sections. Grout the annular space at the end of each day.
- Inspect the installed pipe (man-entry visual inspection)
- Perform leakage or other testing, if required
- Restore flow (remove bypass pumping) if applicable.
- Complete the site restoration and cleanup.
- As necessary, complete the project by constructing or modifying head- and wing-walls on the ends of the culvert.

3.1.9.4_2 Pit Excavation

The jacking unit is substantial and requires a sizeable work pit. The pit size depends on the new pipe material chosen and the length of pipe segments that it needs to accommodate. The jacking unit must have a backstop to push against (Figure 255).

3.1.9.4_3 Pipe Ramming

After the ramming pit is excavated at one end of the culvert, a pipe ramming machine is assembled. A pneumatic hammer is attached to the rear of the steel casing (Figure 254). The rails of the machine are used to support and guide the new casing during the pipe ramming process. A cutting shoe is often welded to the front of the lead casing to help reduce friction and cut through the soil. Bentonite or polymer lubrication can also be used to help reduce friction during ramming operations. An entire length of casing can be installed at once or, for longer runs, one section at a time can be installed. After the casing is in place, the old culvert pipe is removed and all the spoil cleaned out. (The ramming can also be stopped periodically to clear the casing from the enveloped material, if necessary.) A new culvert pipe, e.g., a RCP, is jacked into the casing and the annular space between the new pipe and the casing is filled with grout.

3.1.9.4_4 Pipe Jacking

During pipe jacking, the protective shield (Figure 256) protects personnel and allows them to control line and grade, and the shield also cuts through the soil (if upsizing) shaping the perimeter of the hole opening. The recommended shield length is between 10 ft and 12 ft. Steerable shields that can be steered

by operators inside it are recommended (for better grade maintenance). It is common to use bentonite lubricant to cut down on skin friction around the new pipe (Al Tenbusch, Tenbusch Inc, personal communication).

Figure 254. Pneumatic hammer is attached to the rear of the casing or pipe (Schill, 2007).

Figure 255. Jacking unit inside the pit (Tenbusch and Tenbusch, 2008)

3.1.9.4_5 Replacement with Tunnel Liner Plate

Replacement with tunnel liner plate is appropriate when a jacking pit is not available. The work area must be long enough to launch the shield and to allow for the safe entrance and exit of the workers and materials. A lubricant is not required (Tenbusch and Tenbusch, 2008).

Figure 256. A protective shield attached to a new RCP for protection of workers during removal of CMP and soil excavation (Tenbusch and Tenbusch, 2008)

Figure 257. Two rings of liner plate assembled on the launch track (Tenbusch and Tenbusch, 2008)

3.1.9.5 QA/QC Considerations

The project specifications should include the selection of a contractor with sufficient tunneling experience or require a representative of the equipment manufacturer to be on site. Key items that should be addressed in the contract specifications include pit construction (if applicable), pipe material, grouting, dewatering and ground restoration. Pipe should be manufactured in accordance with applicable ASTM and AWWA standards. Jacking pipe must have acceptable dimensional tolerances for roundness, lengths, end squareness, and straightness, and must have a consistent diameter from pipe to pipe. The jacking pipe must also have flush joints to allow for even transmission of jacking loads, and may need lubrication ports (Al Tenbusch, Tenbusch Inc, personal communication).

3.1.9.6 Standards and Specifications

ASCE 27-00 covers design and installation of precast concrete pipe for jacking in trenchless construction.

ASTM C1208 covers manufacturing, quality assurance testing, inspection, installation, field acceptance testing, and product marking of vitrified clay pipe to be used in microtunneling, pilot tube, sliplining, or pipe bursting.

ASTM D 6783 covers the testing and requirements for polymer concrete pipes. This specification is suited primarily for pipes to be installed by direct burial and pipe jacking.

3.1.9.7 Example Case Histories

In Newark, DE, a failing 48 in. corrugated metal culvert was replaced with a new 54 in. reinforced concrete pipe (RCP) utilizing pipe insertion by pushing. The existing corrugated metal pipe (CMP) had deteriorated seams that allowed the bedding material to migrate into the pipe (Figure 258). The culvert was replaced in both directions from a work pit in the highway median. A shield was attached to a joint of new RCP (Figure 259), which protected the men as they removed the old CMP and excavated the needed amount of material to accommodate the larger diameter pipe. Because the ground was stable, a simple shield was all that was required. Only a small footprint was required to do the work. (Underground Construction, 2009; Tenbusch and Tenbusch, 2008).

Figure 258. Deteriorated CMP culvert (Tenbusch and Tenbusch, 2008)

Figure 259. The traffic along the highway was not affected as the existing 48 in. CMP culvert was replaced with a new 54 in. concrete pipe (Tenbusch et al., 2009).

In High Prairie, Alberta, Canada, 300 miles north of Edmonton, a rotted 36 in. corrugated metal culvert under Highway 2 was replaced by pipe ramming. A 42 in. steel wall carrier pipe (0.562 in. thick) was installed (Werner and Dworsky, 2003).

Five drainage culverts were replaced by ramming between Louisville, KY and Bedford, IN. One of the culverts was a deteriorated 36 in. corrugated metal pipe, through which a small creek ran. The 50 ft of 42 in. casing (0.562 in. thick steel wall carrier pipe) was pounded through limestone floaters, boulders cobble, and railroad ties in less than 90 minutes (Werner and Dworsky, 2003).

3.1.9.8 Advantages and Limitations

Camp et al. (2010) outlined pros and cons of pipe ramming, pipe jacking and tunneling when used for culvert replacement.

Table 31: Pros and cons of pipe ramming, pipe jacking and tunneling (Camp et al., 2010)

	Pros	Cons
Pipe ramming	▪ Can be used for consumption or parallel construction ▪ Works best with smaller diameters ▪ Can eliminate sags in the existing culvert ▪ Does not use a lot of equipment (easy to stop and start if the weather changes) ▪ Allows existing stream to flow through the culvert during construction ▪ Initial work is completed with no personnel entry	▪ Needs to be used in soil conditions ▪ Requires a second liner if the exposed steel is not acceptable ▪ Has to push out old culvert/old materials after the new casing is installed ▪ Cannot see the condition of the backfill materials during the installation process (working blindly)
Pipe jacking	▪ Can be used for consumption or parallel construction ▪ Allows the direct installation of concrete or other pipe material that does not need a secondary lining ▪ Can eliminate sags in the existing culvert	▪ Needs to be used in soil conditions ▪ Requires a large diameter for personnel entry ▪ Needs more equipment, including jacking frame and reaction block
Tunneling	▪ Can be used for consumption or parallel construction ▪ Excavating as you go allows you to see backfill material ▪ Can be used in soil or rock conditions ▪ Can eliminate sags in the existing culvert ▪ Allows for the construction of different shapes of culvert ▪ Allow longer distances to be installed ▪ Permits larger diameter openings	▪ Needs more equipment; therefore difficult to stop and start ▪ Requires a larger diameter for personnel entry ▪ Requires a second liner if liner plate or steel ribs are used

3.1.10. SPOT REPAIRS

3.1.10.1 Introduction

Spot repairs involve a variety of methods that repair relatively short sections of culvert, defective joints or defective lateral connections (Sterling et al., 2009), e.g., chemical grouting, robotic repairs, spot repair sleeves (mechanical), and internal joint sealing.

A chemical grout is injected under pressure through cracks and defective joints into the surrounding soil to seal the defective pipe and protect against infiltration/inflow. Although resulting in a localized repair, the method can be used to seal the entire pipe by repeating the test-and-seal procedure along the pipe.

Robots are used for mechanical work (e.g., cutting off or grinding protruding laterals, grinding of damaged pipe to expose virgin pipe material) and for applying a repair material (resin or mortar) without or with pressure to the prepared surface. If pressure is applied, the material penetrates the soil behind the defective pipe where a sealing collar is created around the pipe. Robotic repairs are carried out as spot repairs anywhere in culverts or at lateral connections.

Spot repair sleeves – Prefabricated stainless steel or PVC sleeves are positioned inside the pipe while folded and then jacked (snapped) into an expanded shape. The sleeves offer a structural repair of the damaged pipe but can also restore missing pipe sections without excavation or provide infiltration sealing of joints. Although multiple sleeves can be used adjacently, this method is typically used to repair relatively short sections of pipes.

In addition, sectional CIP liners have been installed in culvert pipes, which provide a high strength structural repair.

3.1.10.2 Chemical Grouting

3.1.10.2_1 Overview

Chemical grouting is used for sealing leaking joints or cracks in concrete pipes, bolt holes in corrugated metal pipes, etc. Chemical grouting can be performed by applying several different techniques:

- Test-and-seal procedure - A short section of pipe is isolated and pressure-tested for leak tightness. If the test fails, the segment is sealed with self-setting chemical grout. The grout is injected under pressure to pass through the pipe wall into the surrounding soil where it mixes with the soil creating a sealing collar of material around the pipe (Figure 260).
- Expanded gasket placement (EGP) technique - A resin soaked rod (an open cell poly foam round rod) or oakum is inserted into the moistened joint (Figure 261) and firmly packed, which is followed by the grout compound expanding in contact with water and curing to form a cellular rubber gasket.
- Pressure injection - Injection holes are drilled in the concrete wall along the face of the crack at an angle to intercept the crack and injectors are staged to fill the crack with grout (Figure 262).

Figure 260. Sealing collar of material around the pipe made of chemical grout and soil (Henning, 2002)

Figure 261. Resin soaked rod being inserted into the joint as part of expanded gasket placement (EGP) technique (Henning, 2003)

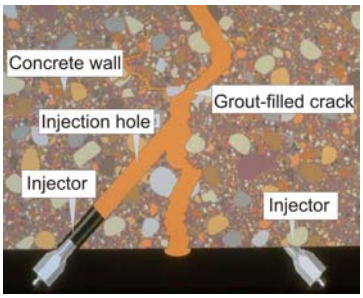


Figure 262. Pressure injection of grout through injection holes drilled in concrete (Henning, 2003)

With the exception of EGP technique, chemical grout injected under pressure typically passes through the pipe wall into the surrounding soil, where it mixes and bonds with the soil and creates a sealing collar of material around the pipe (Figure 260)

3.1.10.2_2 Materials Used

Commonly used chemical grouts include acrylamides, acrylates, polyurethanes, hydrophilic expansive grouts, hydrophobic expansive grouts, ultrafine cementitious grouts, and epoxies (Magill and Berry, 2007).

3.1.10.2_3 Applicability

Chemical grouts do not provide a structural repair and the method is applicable only in structurally sound pipes.

Test-and-seal procedure can be performed in pipes ranging in circular diameter from 6 in. to 144 in. Packers are available for other pipe shapes, e.g. box culverts (Figure 263) or elliptical. For chemical grouting of large diameter corrugated metal pipes (24 in. and up to 96 in.), collapsible packers are used, which have a soft natural rubber sleeve covering the two end bladders to seal against rough surfaces (Figure 264) and can be dismantled for introduction through accesses as small as 21 in. in diameter (Figure 265). Test-and-seal procedure sometimes cannot be applied (the isolated section cannot be pressurized due to too many voids in the soil envelope around the pipe).

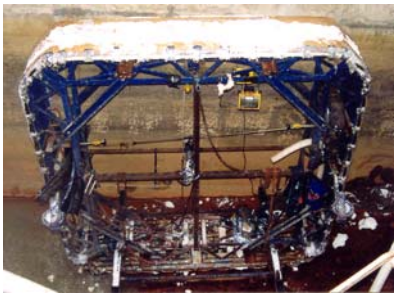


Figure 263. Custom designed packer for test-and-seal procedure in a 10 ft×10 ft box culvert (Marc Anctil, Logiball, Inc, personal communication)

Figure 264. Collapsible packer for a large diameter CMP culvert pipe (Marc Anctil, Logiball, Inc, personal communication)

Figure 265. Collapsed packer with large wheels for easy movement (Marc Anctil, Logiball, Inc, personal communication)

3.1.10.2_4 QA/QC Considerations

NASTT (2006c) outlined a quality assurance plan for chemical grouting: follow good specifications (samples available from manufacturer, NASSCO publications), CCTV inspect the pipe (to verify cleanliness, condition and sealing), test all joints in a section, and verify grout consumption relative to a value obtained using a standard rule of thumb (provided by the manufacturer). Quality control measures include counting pump strokes (14 strokes = 1 gallon grout pumped), verifying levels in grout tanks (must be equal with 1:1 mix ratio for materials), limit maximum production in an 8-hour day (e.g., 125 joints in an 8 in. pipe), and take advantage of a 12-month contractor warranty (re-test prior to end of the warranty period).

3.1.10.2_5 Standards and Specifications

ASTM F2304 covers a procedure to inspect, test, and seal sewer pipe joints, having selected the appropriate chemical grouts, using the packer method. This practice should not be used for longitudinally cracked pipe, severely corroded pipe, structurally unsound pipe, flattened, or out-of-round pipe.

ASTM F2414 provides a guide for the installation of chemical grout in the practice of sealing sewer manholes in the case of leaks, and cracks.

3.1.10.2_6 Example Case Histories

In Denver, CO, a precast concrete storm sewer pipe, 7 ft in diameter, was rehabilitated with chemical grouting in 1995. The pipe contained 139 bell-and-spigot joints, between 0.25 in. and 2.25 in. wide, with original gasket material missing at certain locations and presumably some voids in the soil behind them. A single component hydrophilic polyurethane was used. The joints were first treated with resin saturated oakum (EGP technique), and subsequently the urethane grout was pumped through holes drilled about 4 in. from each joint at an angle to intercept the joint behind the activated oakum (pressure injection). The water-to-resin ratio of the grout was adjusted on site from initial 5:1 to 1:1, because the volume of grout being pumped initially was up to six times the theoretical value. The change increased grout viscosity and reduced its setup time (at 60°F, it drops from 345 seconds to 90 seconds) and remedied the problem (Gumina, 1995).

In Hazelwood, MO, chemical grouting was used to seal joints in 10ft×10ft box culvert that had opened when the culvert initially settled allowing water and sand to enter the culvert, which caused further settlement. The joints on the 7 ft long sections were grouted, sealing and stabilizing over 1,700 ft of culvert. Acrylamide grout was pumped under pressure (20 psi) and some joints took as much as 200

gallons (Thomas, 1999).

3.1.10.2_7 Advantages and Limitations

The main advantages of chemical grouting are the ability to stop infiltration/exfiltration quickly and to stabilize the soil bedding surrounding the pipe without the need for excavation or surface disruption. With test-and-seal procedure, the repair is performed only where it is needed. The method is generally less expensive than other rehabilitation methods.

The main limitation of this method is that structural repair is not provided. The longevity of repair appears to be shorter than with other trenchless rehabilitation methods: short life of installed chemical grout was reported in some case studies (e.g., between 2 and 5 years) although good performance of installed grouts was reported in other case studies (10 to 20 years after the rehabilitation).

3.1.10.3 Mechanical Repair Sleeves

3.1.10.3_1 Overview

With this method, prefabricated stainless steel or PVC sleeves are positioned inside the pipe while folded and then jacked (snapped) into an expanded shape (Figure 266). The annular space between the sleeve and the pipe is filled with grout. The sleeves offer structural repair to damaged pipes, but can also restore missing pipe sections without excavation and seal the joints against infiltration. The method is effective (Ballinger and Drake, 1995) in re-rounding distorted corrugated metal pipe sections.

Figure 266. Installing a PVC sleeve into a damaged HDPE pipe (LINK-PIPE, 2008d)

Mechanical repair sleeves come in various diameters and short standard lengths (e.g., 18 in., 24 in., or 36 in.).

3.1.10.3_2 Materials Used

Sleeves made of stainless steel or rigid PVC can be used. Polyurethane grout is used to fill the annular space between the sleeve and host pipe when the repaired pipe is required to retain flexibility. Cementitious grout offers an economic alternative when joint flexibility is not required.

3.1.10.3_3 Applicability

PVC sleeves are applicable in man-entry culvert pipe ranging in diameter from 36 in. to 108 in. Both circular and non-circular pipes (tear-drop, horse-shoe and oval pipes) can be repaired. The sleeves made of rigid PVC can be adapted to most culvert designs except box culvert.

Stainless steel sleeves are applicable in pipes ranging in diameter from 6 in. to 54 in.

3.1.10.3_4 Construction Details

The general installation procedure of PVC sleeves (Figure 267) involves the following steps (LINK-PIPE, 2008d):

- Position the sleeve inside the pipe to cover either the damaged area or, if used for joint sealing, center it on the joint
- Expand the sleeve using two hydraulic jacks
- Grout the annular space

Two hydraulic jacks (rams) are used for sleeve expansion: the first is used to hold the snap-out sleeve vertically against the top and bottom of the pipe; and the second is positioned horizontally to snap-out the flaps at the springlines and lock them into position (Figure 268). For grouting, liquid grout is pumped through the vent nipple at the invert and into the annular space until dense foam, or liquid grout, starts emerging from the crown vent (Figure 269).

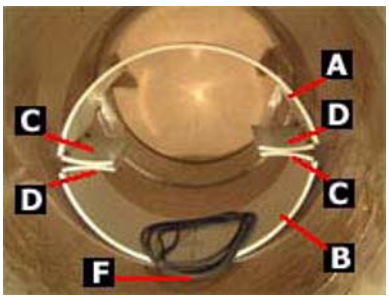


Figure 267. PVC repair sleeve consisting of six segments (LINK-PIPE, 2008d)

Figure 268. Jacks used for sleeve expansion (LINK-PIPE, 2008d)

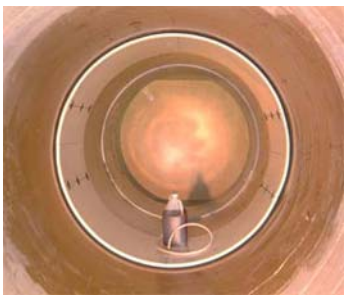


Figure 269. Grouting of annular space (LINK-PIPE, 2008d)

The general installation procedure of stainless steel involves the following steps (Ballinger and Drake, 1995):

- Insert the sleeve into the culvert
- Insert the pneumatic plug into the sleeve and slightly inflate it to hold the sleeve
- Pull the sleeve/plug assembly to the location that needs repair
- Inflate the pneumatic plug to expand the steel sleeve and compress the polyethylene gasket
- Deflate the plug so that the edges of the sleeve snap together
- Grout the annular space

3.1.10.3_5 Example Case Histories

In Atlanta, GA, a 60 ft long corrugated metal culvert, 47 in. in diameter, was repaired with a PVC sleeve, 45.5 in. in diameter, in 2007 (LINK-PIPE, 2008a).

In Pentagon City, VA, a damaged HDPE culvert was repaired with a mechanical PVC repair sleeve (LINK-PIPE, 2008a).

3.1.10.3_6 Advantages and Limitations

The main advantage of mechanical repair sleeves is the ability to provide a structural repair with a very quick and simple procedure. Furthermore, no excavation is required, nor is expensive installation equipment, there are short setup and installation times, and a small crew (three people) is needed that can be trained in one day.

3.1.10.4 Sectional Cured in Place Liners

3.1.10.4_1 Overview

Sectional cured-in-place liners are short CIP liners (CIP sleeves) that can be installed by the following methods (Flanery and Kiest, 2004):

- Inversion method – The liner tube, contained within the inversion bladder and vacuum impregnated with resin, is loaded into a flexible launching device. After positioning at the beginning of the damaged pipe section, the liner is inverted inside out forcing the resin against the pipe wall. After resin cure, the bladder is re-inverted, and is thus peeled away from the cured liner.
- Packer wrapping method – The liner flat sheet is manually impregnated with resin and wrapped tightly around a standard sewer plug overlapping the ends to produce the full circle tube. The liner/plug assembly is pulled through the pipe until the assembly reaches the point of repair. The packer method is not contained and a significant resin loss occurs during the pull. The plug is inflated and held in place during the resin cure. The apparatus is deflated relieving the air pressure and removed from the pipe.
- Pull in and inflate method – The liner tube with a protective coating on the outside and a bladder hose inverted inside it, is resin impregnated and pulled through the pipe until the assembly reaches the point of repair. The bladder is inflated and held in place during the resin cure, and thereafter re-inverted so it peels away from the cured liner. The protective coating on the outside prevents resin migration and bonding with the host pipe.

The final product can stop infiltration, eliminate root intrusion, is chemically resistant, and can provide a full structural repair (it can bridge missing pipe sections).

3.1.10.4_2 Materials Used

Sectional CIP liners use the same materials as full length CIP liners. Tube material can be one or more layers of flexible needled felt or an equivalent non-woven material, and resin is polyester or vinyl ester with proper catalysts as designed for the specific application.

One system utilizes a tube consisting of an inner liner (non-woven flexible needled felt), a protective coating (PU/PVC), and a fiberglass/felt mat reinforcement (an additional layer of chopped fiberglass and felt). An epoxy resin is formulated and applied to the inner liner, as well as to the fiberglass/felt mat. The inner liner and fiberglass/felt mat become one with the impregnation of the epoxy resin (Cole Perkins, Perma-Liner Inc, personal communication).

3.1.10.4_3 Applicability

The sectional CIP sleeves typically repair short sections of pipe between 2 ft and 30 ft and are installed in pipes ranging in diameter from 6 in. to 54 in. They are installed in corrugated metal pipes as well as other culvert pipe materials.

3.1.10.4_4 Standards

ASTM F2599 covers requirements and test methods for the sectional cured-in-place lining (SCIPL) repair of a pipe line (4 in. through 60 in.) by the installation of a continuous resin-impregnated-textile tube into an existing pipe by means of air or water inversion and inflation. The tube is pressed against the host pipe by air or water pressure and held in place until the thermoset resins have cured (Installation standard).

3.1.10.4_5 Advantages and Limitations

The main advantage of sectional CIP relining is the ability to provide a structural repair of a CIP liner with reduced cost. No flow isolation or by-pass is required in many cases (the bladders have a 2 in. flow-through running through them to alleviate upstream head pressure). The installation is relatively quick (approximately 3.5 hours per repair section) and no digging is required.

The main limitation is that the method is not applicable in pipes with severe mineral buildup, severe offset joints, or sags.

3.1.10.5 Internal Joint Sealing

3.1.10.5_1 Overview

Internal joint sealing is the method in which a mechanical leak clamp consisting of a rubber seal is installed on the inside of the pipe to provide sealing against the pipe wall on either side of the joint (Figure 270) (Cronin, 1988). The seals create non-corrosive, water-tight connections around the full inside circumference of the pipe at the joint area. Figure 271 shows the seals installed over joints in a corrugated metal pipe. The seals can also be installed with overlap, thus creating an interlocking sleeving system (Figure 272).

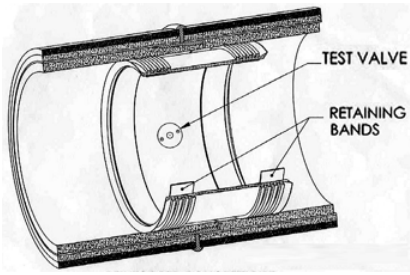


Figure 270. Rubber seal spanning the joint is held in place by stainless steel retaining bands on both sides of the joint (Cronin, 2007)

Figure 271. Internal joint seals installed in 36 in. corrugated metal storm drain (Cronin, 2007)

Figure 272. Installation of interlocking sleeving system (Cronin, 2007)

3.1.10.5_2 Materials Used

The seals used in culvert pipes are made of EPDM rubber and the retaining bands from stainless steel.

3.1.10.5_3 Applicability

The method can be utilized in pipes from 16 in. in diameter up to 216 in. (Cronin, 1988). Seals have been installed in different pipe types, e.g., concrete, reinforced concrete, steel, VCP, and plastic pipes (PVC and HDPE). In addition to round seals, other shapes of internal joint seals (e.g., elliptical, arched) can be custom fabricated.

3.1.10.5_4 Installation Procedure

The hydraulic jack installation method for internal joint seals includes the following steps (Cronin, 1988):

- Prepare the joint (clean and if necessary fill it with mortar to render the joint flush with the pipe wall and to provide a firm solid backing)
- Position the seal centering it over the joint

- Position retaining bands in the seal and expand each band
- Pressure test the installed seal
- Check for leaks (a soap solution is sprayed)

Expansion of the bands can be performed in two different ways:

- Pressurized air is introduced and maintained for a required time (at least two minutes)
- Tightening the bolts on wedge lock assemblies that are attached to the expansion bands increases the effective diameter of the bands thus expanding the bands (Figure 270). Depending on the diameter of the pipe joint being sealed, there is typically between one and four WedgeLock Assemblies on each band (Trelleborg Pipe Seals, 2009).

Figure 273. Mechanism of increasing the expansion band's diameter by tightening a bolt on an attached Wedge Lock assembly (Trelleborg Pipe Seals, 2009)

3.1.10.5_5 Example Case Histories

Oregon DOT used both internal joint seals and CIP relining to rehabilitate a 280 ft long 48 in. pipe (fiberglass pipe, sewer) in Clatskanie, OR in 2000. The upstream portion was in good condition except for two cracked joints rehabilitated with seals but the bottom 180 ft of pipe was in very poor condition and was rehabilitated with CIPP (George Vernon, Mill Creek Management Technologies, personal communication).

Oregon DOT used internal joint seals for repair of corrugated metal culverts in 2007. In Banks, OR, a 48 in. round culvert 100 ft long (Hwy 6 and Hwy 26, Bauch Creek Irrigation Culvert Project), in good overall condition, nevertheless had two failed joints experiencing infiltration that were repaired installing 47.5 in. diameter internal seals (George Vernon, Mill Creek Management Technologies, personal communication).

3.1.10.5_6 Standards

ASTM C150 covers Portland cements to be used for preparation of pipe joint sealing mortars.

3.1.10.5_7 Advantages and Limitations

The main advantage of internal joint seals is the ability to provide a flexible, watertight seal for leaking joints with a quick and simple procedure. No excavation is required, there is no expensive installation equipment, and the procedure requires short setup and installation times.

The main limitation is that the repair is limited to the joints and all other defects in the pipe need to be addressed with other rehabilitation methods. Durability could be questionable in some parts of the country (rodents like nutria eat the material, Mike Dunning, ORDOT, personal communication).

3.1.11. INVERT REHABILITATION

3.1.11.1 Invert Paving with Concrete

One of the most effective ways to rehabilitate corroded and severely deteriorated inverts in corrugated metal pipes is paving them with concrete. With certain modifications, the method can be used in concrete culverts. The method consists of pouring a concrete lining in the culvert invert, which increases surface roughness and thus decreases flow velocity and protects the culvert against abrasion. The method is usually limited to large diameter culverts (120 in. or more), and is generally applied on steep slopes that operate under inlet control (Caltrans, 2003). Portland cement based concretes including conventional concrete, high-strength concrete, or steel fiber reinforced concrete can be used (Ballinger and Drake, 1995).

Ballinger and Drake (1995) abstracted procedures for invert paving with concrete, which have been originally provided in NCSPA (1988). In metal culverts, the following are their guidelines:

- Before paving, a flow diversion (bypass) is conducted and the culvert surface to which the concrete will be applied is prepared (must be clean and dry). Any debris, dirt and other material is removed from the culvert pipe. Any void areas are filled with concrete (portions of the culvert's invert are removed as necessary to enable filling of the voids).
- Class C concrete can be used using the following mixture: 1,440 lbs/cu.yd. fine aggregate; 1,410 lbs/cu.yd. coarse aggregate (#8 max stone); cement 600 lbs/cu.yd.; water/cement ratio 0.5.
- The bottom portion of the pipe is paved as follows:
    - In round pipes, the bottom 25% of the inside pipe circumference,
    - In pipe arches with spans of up to 10 ft 3 in., the bottom 30% of the inside periphery
    - In pipe arches with spans over 10 ft 3 in., the bottom 35% of the inside periphery
- The minimum coverage is 4 in. over corrugations.
- The pavement is reinforced with steel fabric reinforcement meeting specifications in FHWA's Standard Specifications (FHWA, 2003: Section 709 Reinforcing Steel and Wire Rope). The reinforcement can be installed in the following alternate methods:
    - A wire, No. 6 gauge, on 6 in. centers longitudinally and transversally, is placed 2 in. above the corrugations. The wire is attached to the pipe by welding to the crest of corrugations or tied to studs or angles previously welded to the pipe (spaced 2 ft apart longitudinally and transversally).
    - A wire mesh, 6×6×8×8 (6 in.×6 in.×2.1 mm×2.1 mm), is positioned so that transverse wires lie in the groves of the multi-plate corrugated sheets. The wire mesh is tack welded to invert every 30 in. longitudinally and every 2 in. transversally.
- The concrete surface is protected within 18 hours after completion or, if exposed to sunlight, immediately after completion. For protection, approved curing cover or membrane curing compound applied at a min rate of 1 gallon per 150 sq. ft. can be used.
- The concrete is left to cure for a minimum of 48 hours before the water is permitted to flow inside the culvert.

Ballinger and Drake (1995) also referenced a specification by the US Army Corps of Engineers for invert paving in a concrete culvert with severe abrasion problem. Two types of concrete were specified as alternatives, and a method of bonding the new invert concrete with the existing concrete culvert pipe:

- Steel fiber reinforced concrete can used, composed of 3/8 in. coarse aggregate, fine aggregate, Portland cement, wire fibers, water and admixtures (batched and mixed in accordance with ASTM C94). Ferrous wire fibers can be round, flat or deformed, 1 in. long.
- High strength concrete can also be used as an alternative for fibrous reinforced concrete.

- Steel reinforcing bars are used to anchor the fibrous concrete to the invert of the pipe. The bars are straight and hooked on one end. Holes for anchor bars are drilled at specified angle in the culvert invert (hole diameter no less than 1.5 times the diameter or greatest transverse dimension of the anchor bar). The holes are filled with grout the anchor bars forced into place and vibrated, and the grout left to cure for at least 36 hours before applying the fibrous concrete.

Moll (1993) reported ten corrugated metal culvers rehabilitated with invert paving by the Oklahoma DOT. Two different concrete design mixes were used and both provided “very good, no cracking” performance ratings. The cement grout was manually trowelled to create 3 in. thick layer above the corrugations. The width of the paving was 75% of the pipe diameter.

3.1.11.2 Invert Lining with Steel Armor Plates

The method consists of installing steel plates in the culvert invert area, which provides a smooth surface and reduced impediment to the flow compared to the corrugated surface, thus ultimately reducing the effect of abrasion. Steel plates are between 0.37 in. and 1 in. thick (Caltrans, 2003).

Figure 274. Invert steel armor plating (Caltrans, 2003)

Figure 275. Finished steel plated invert (Caltrans, 2003)

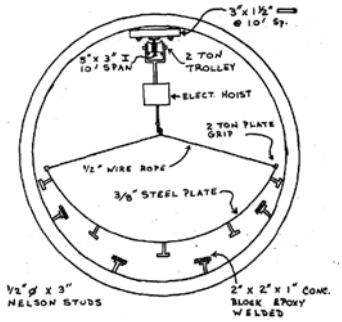


Figure 276. Installation of steel plates (Ikerd, 1984)

Ikerd (1984) reported the use of steel armor plates by Alabama DOT to repair corrugated metal inverts severely deteriorated by abrasion. The project, completed in 1974, repaired the inverts of a triple barrel culvert. Each barrel was 84 in. in diameter and 517 ft long.

Ikerd (1984) provided construction details about this method. Anchors are welded to the existing pipe inverts and a concrete mortar bed placed along the pipes. Next, the steel plates are lowered vertically into the mortar bed. An overhead monorail system attached to the crown of the pipe was used for transporting the steel plates (Figure 276).

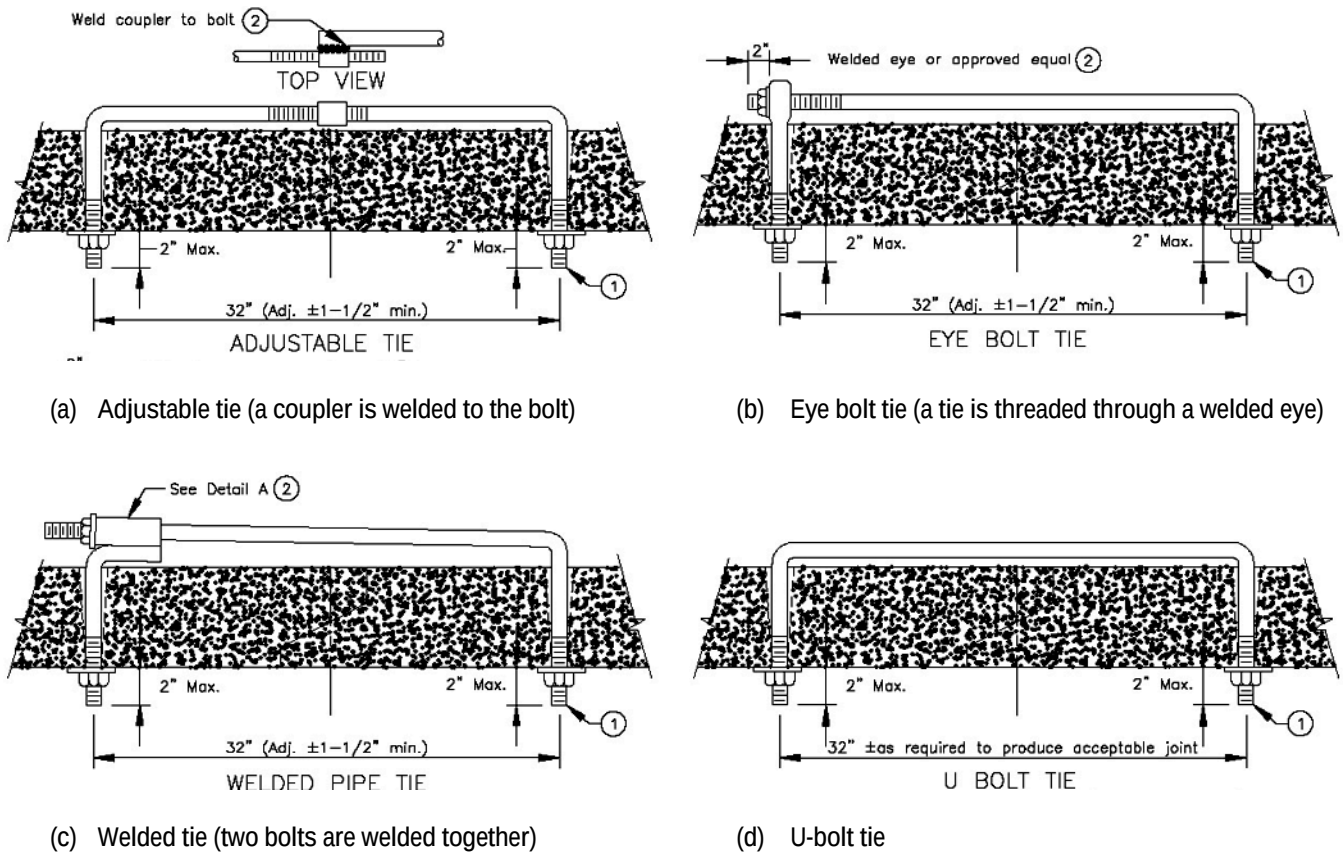
3.1.12. REINFORCING CULVERT END SECTIONS

End sections without adequate soil support can be reinstated by underpinning. Ballinger and Drake (1995) described concepts and methods for underpinning in Appendix B-20.

The following methods are suitable for reinforcing end sections of concrete culverts to prevent their drop off:

- Concrete pipe ties
- Steel reinforcing bars
- Pre-stressing steel

Concrete pipe ties can be used in new installations. Four different methods to attach concrete pipe ties are shown in Figure 277.



(a) Adjustable tie (a coupler is welded to the bolt)

(b) Eye bolt tie (a tie is threaded through a welded eye)

(c) Welded tie (two bolts are welded together)

(d) U-bolt tie

Figure 277. Four methods that can be used to attach concrete pipe ties in new culverts (MNDOT, 1985)

Steel reinforcing bars have U-shaped end sections that are installed into the holes pre-drilled through the concrete sections. The bars may be anchored with an epoxy adhesive. The method is applicable in man-entry culverts and is suitable for culverts where the embankment cannot be removed (Ballinger and Drake, 1995).

Ballinger and Drake (1995) introduced the possibility of pre-stressing the steel in the reinforced concrete culvert to keep the sections together and prevent them from separating. No case study has been identified. The pre-stressing would have to be done on the entire length of the culvert and not just on the end sections.

3.1.13. APPURTENANCES

3.1.13.1 Overview

This section reviews adding new appurtenances that can structurally reinforce the culvert barrels and/or enhance their hydraulic performance, and procedures for the repair of damaged appurtenances.

3.1.13.2 Endwalls and Wingwalls

Endwalls and wingwalls can reinforce the culvert barrel, protect it against the erosion, and act as a counterweight to offset buoyant forces (Figure 278). Installing a new headwall may be more efficient than culvert replacement when hydraulic capacity of the existing culvert needs to be increased.

Wingwalls help mold and direct channel flow into the culvert and protect the area around the inlet from scour. Endwalls are usually constructed parallel with the embankments at the ends of the culvert (Figure 279) (Ballinger and Drake, 1995).

Deteriorated or collapsed endwalls and windwalls can be replaced. Ballinger and Drake (1995) provided guidelines for replacing headwalls and windwings in Appendix B-16, and for repairing basically sound headwalls and windwings in Appendix B-17. Procedures for repairing severely deteriorated headwalls and windwings are listed in Appendix B-18.

Figure 278. Headwall and wingwalls (CDOT, 2004)

Figure 279. Culvert headwall parallel with the embankment (Kearley and McCallister, 2000)

3.1.13.3 Energy Dissipators

Energy dissipaters reduce the flow velocity in a culvert and thus abrasion related wear of culvert. General types of energy dissipaters include hydraulic jump, forced hydraulic jump, impact, drop structure, stilling well, and riprap.

FHWA's HEC 14 (Thompson and Kilgore, 2006) described various energy dissipaters, including their design, applicability and limitations.

Energy dissipaters can be external (located outside of the culvert barrel) and internal (located within the culvert barrel). Internal dissipators are intended to form the hydraulic jump within the culvert, thus eliminating costly outlet structures. One type of internal energy dissipators are circular rings spaced along the pipe at the downstream end (Figure 280), described in HEC 14 (Thompson and Kilgore, 2006), and Wiggert and Erfle (1972).

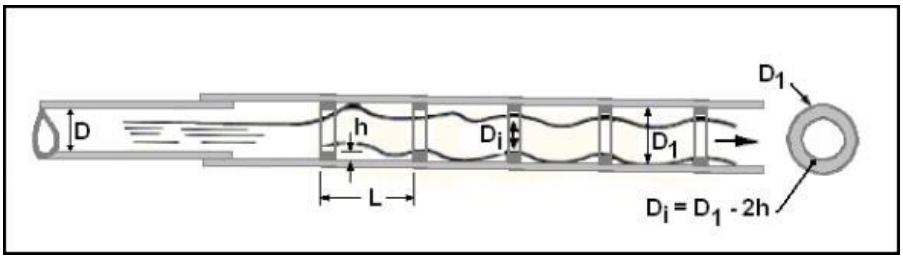


Figure 280 Circular rings inside the barrel (Thompson and Kilgore, 2006)

One type of external energy dissipators are stilling basins, characterized by some combination of chute blocks, baffle blocks, and sills designed to trigger a hydraulic jump in combination with a required tailwater condition (Figure 293).

Drop structures are commonly used for flow control and energy dissipation. Changing the channel slope from steep to mild, by placing drop structures at intervals along the channel reach, changes a continuous steep slope into a series of gentle slopes and vertical drops. Instead of slowing down and transferring high erosion producing velocities into low non-erosive velocities, drop structures control the slope of the channel in such a way that the high, erosive velocities never develop. A grate or series of rails forming a

"grizzly" may be used in conjunction with drop structures (Figure 294). The incoming flow is divided into a number of jets as it passes through the grate (Thompson and Kilgore, 2006).

Stilling wells can be used in channels with moderate to high concentrations of sand or silt and where debris is not a serious problem (Thompson and Kilgore, 2006).

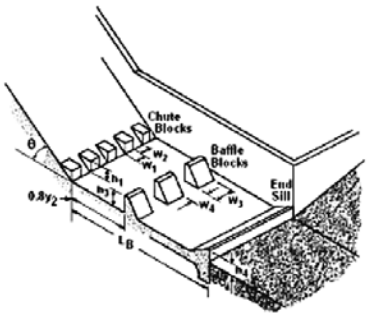


Figure 281. USBR type III stilling basin (Thompson and Kilgore, 2006)

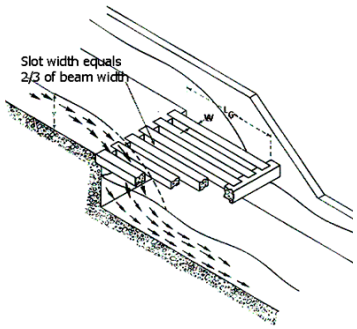


Figure 282. Drop structures (Thompson and Kilgore, 2006)

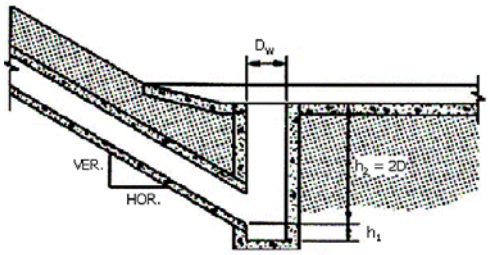


Figure 283. Stilling well (Thompson and Kilgore, 2006)

Riprap basin energy dissipators can be based on armoring a pre-formed scour hole (Figure 284) or they can be made as riprap aprons that provide a flat armored surface. Both types were developed by U.S. Army Corps of Engineers (Bohan, 1970; Fletcher and Grace, 1972). Riprap energy dissipators are adaptable to regions where riprap in the required sizes, gradation, and quantity is readily and economically available.

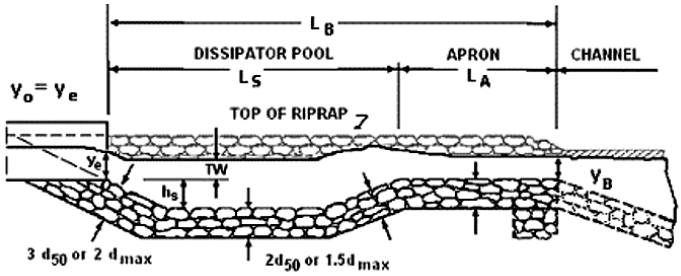


Figure 284. Riprap basin energy dissipators (Thompson and Kilgore, 2006)

Figure 285. Riprap at culvert outlet (Watershed Steward Demonstration Site Examples, downloaded www.livestockandland.org/Demonstration_Sites/pilot_site.html on June 17, 2010)

Limitations for different dissipator types are summarized in Table 32, which can be used to determine what alternative types to consider in particular situations (CDOT, 2004). Various energy dissipaters and stilling basins are described in other publications, e.g., City of Knoxville (2010).

Table 32: Limitations for different energy dissipator types (after CDOT, 2004)

Internal dissipators	Natural scour holes	External dissipators	Stilling Basins
▪ The scour hole at the culvert outlet is unacceptable ▪ The right-of-way is limited ▪ Debris is not a problem; and ▪ Moderate velocity reduction is needed	▪ Undermining of the culvert outlet will not occur or it is practicable to be checked by a cutoff wall ▪ The expected scour hole will not cause costly property damage; and ▪ There is no nuisance effect.	▪ The outlet scour hole is not acceptable ▪ Moderate amount of debris is present; and ▪ The culvert outlet velocity is moderate and corresponding Fr < 3.	▪ The outlet scour hole is not acceptable; ▪ Debris is present; and ▪ The culvert outlet velocity is high, and corresponding Fr > 3.•

3.1.13.4 Aprons and Scour Protection

Scouring of culvert outlets is a common problem that can lead to damage to the culvert structure and neighboring land. Scour is possible at both culvert inlet and outlet.

Mendoza et al. (1983) found that the maximum depth, width, length, and volume of scour can be correlated to the discharge intensity but also observed that the maximum scour dimensions are approximately equal for both headwall and no-headwall conditions. Abida and Townsend (1991) investigated the local scouring phenomenon in sand that occurs downstream of box culverts. The principal factors governing this form of local scouring were found to be the discharge rate, the culvert width, the tailwater depth, the downstream channel width, and the bed-material properties. Liriano et al. (2002) measured the turbulent flow structure within scour holes downstream of pipe culvert outlets at different stages of development. Initial scour hole development was found to be a result of high velocities exceeding the critical velocity for sediment transport whilst further development results in a reduction in the magnitude of the near-bed velocities and an asymptotic increase in scour depth associated with the turbulent structure of the flow. Towards the downstream end of the scour holes, the jet comes into contact with the bed and flow structures similar to those observed downstream of backward facing steps are noted. FHWA's HEC 14 (Thompson and Kilgore, 2006) presented a method for predicting local scour at the outlet of culverts based on discharge, culvert shape, soil type, duration of flow, culvert slope, culvert height above the bed, and tailwater depth. The equations were derived from tests conducted by the Corps of Engineers (Bohan, 1970) and Colorado State University.

FHWA's HEC 23 (Lagasse et al., 1997) provided guidelines for the selection and design of appropriate scour countermeasures to mitigate potential damage to bridges and other highway components at stream crossings.

Gannett (2008) outlined the following culvert scour countermeasures:

- Inlet and outlet cutoff walls
- Stone fill apron at outlet
- Headwalls/wingwalls to protect embankment
- Downstream check dam where stream degradation exists
- Pre-formed scour hole for energy dissipation and fish habitat

NYSDOT (2005) Highway Design Manual (HDM) recommends culvert cut-off wall for all reinforced concrete box culverts with invert slab to prevent undermining as follows:

- Minimum width 450 mm (18 in.),
- Extend 1.2 m (47 in.) below invert, or to sound rock if shallower,
- Wall at both inlet and outlet ends,
- An additional cut-off wall should be specified at the end of the apron when a concrete apron is specified (however, if the apron is continuous with the barrel, cut-off wall is only required at the end of the apron); and,
- Wingwall footings should be set at or below bottom of cut-off wall.

Stone or concrete aprons can be used for scour protection of culverts. NYSDOT (2005) also recommends placing a stone apron at the outlet and at the inlet of all culverts as follows:

- Minimum length of the stone apron 7.5 m (24 ft).
- The stone apron should cover the full width of the streambed, and in addition, stone filling should be placed on the side slopes to a minimum elevation of 300 mm (1 ft) above Design High Water.
- A 1.5 m wide by 1.2 m deep (5 ft by 4 ft) key of stone filling should be placed in the streambed at the end of the apron away from the culvert.
- Stone filling should also be used to stabilize all disturbed slopes to a minimum elevation of 300 mm (1ft) above Design High Water.

Figure 286. Culvert stone apron (Gannett , 2008)

Ballinger and Drake (1995) described several types of riprap (rock riprap, gravel riprap, wire-enclosed riprap), the procedures of installing them (Appendix B-13) and the procedure for the repair and replacement of culvert concrete or masonry aprons (Appendix B-14).

3.1.13.5 Debris Control Structures

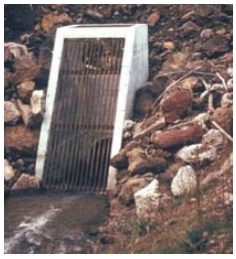
FHWA's HEC 9 (Bradley et al., 2005) described problems associated with debris accumulation at culverts, provided guidelines for analyzing and modeling debris impacts on structures, and presented general criteria for selecting and designing debris countermeasures. The accumulation of debris at inlets of highway culverts may result in erosion at culvert entrances, overtopping and failure of roadway embankments and damage to adjacent properties, increased local scour at piers and/or abutments, and the formation of pressure flow scour. Various types of structural measures are shown in Table 33 and in Figure 287.

Table 33: Structural debris control measures for culverts (after Bradley et al., 2005)

Measure	Description
Debris deflectors	Structures placed at the culvert inlet to deflect the major portion of the debris away from the culvert entrance. They are normally "V"-shaped (a).
Debris racks	Structures placed across the stream channel to collect the debris before it reaches the culvert entrance. Usually vertical and at right angles to the streamflow, but they may be skewed with the flow or inclined with the vertical (b).
Debris risers	Closed-type structure placed directly over the culvert inlet to cause deposition of flowing debris and fine detritus before it reaches the culvert inlet. Risers are usually built of metal pipe
Debris cribs	Open crib-type structures placed vertically over the culvert inlet in log-cabin fashion to prevent inflow of coarse bed load and light floating debris
Debris fins	Walls built in the stream channel upstream of the culvert. Their purpose is to align the debris with the culvert so that the debris would pass through the culvert without accumulating at the inlet (c).
Debris dams and basins	Structures placed across well-defined channels to form basins which impede the stream flow and provide storage space for deposits of detritus and floating debris (d).
Combination devices	Combination of two or more of the preceding debris-control structures at one site to handle more than one type of debris and to provide additional insurance against the culvert inlet from becoming clogged.

Non-structural debris control measures include emergency maintenance (removing debris from the culvert entrance and/or an existing debris-control structure) and annual maintenance (removing debris from within the culvert, at the culvert entrance, and/or immediately upstream of the culvert, or repairing any existing structural debris control measures).

(a) Steel rail and cable debris deflector.



(b) Debris rack

(c) Concrete debris fin

(d) Debris dam of precast concrete sections

Figure 287. Examples of structural debris countermeasures (after Bradley et al., 2005)

Table 34 provides a debris-control countermeasures matrix intended to guide the user in the selection of countermeasures.

Table 34: Culvert debris-control countermeasures matrix (after Bradley et al., 2005)

Counter-measures	Debris Classification							Maintenance	Aesthetics	Environ. Impact	Installation Experience by State	Design Guideline
	Floating Debris			Flowing Debris	Bed Material							
	S	M	L		F	C	B					
Structural countermeasures												
Deflectors		x	x				x	H	A	L	CA	6.2.1
Racks	x	x						H	A	L	CT, CA	6.2.2
Risers				x	x	x		L	A	L	CA	6.2.3
Cribs	x					x		M	A	L	CA	6.2.4
Fin			x					M	A	L	SD, TN, CA	6.2.5
Dams, Basins				x	x	x		H	A	H	Widely Used	6.2.6
Non-structural countermeasures												
Emergency and annual maintenance		x	x	x				H	U	M	Widely Used	-
Debris management plan		x	x	x				H	D	L		6.4
	S=small M=medium L=large			F= fine detritus C= coarse detritus B= boulders				H=high M=moderate L=low	A=acceptable D=desirable U=undesirable	H=high M=moderate L=low		Section in FHWA's HEC 9

Ballinger and Drake (1995) provided guidelines for debris removal (Appendix B-1) and sediment removal (Appendix B-2). A high pressure water hose and vacuum systems offer cost-effective, efficient ways to flush sediment from culverts. Vacuum cleaning (Figure 288) is a cost effective method of removing dirt, grit, and other debris from culverts, and can be used for removal of practically any type of material (earth, water, sludge, spills, debris, etc.).

Figure 288. Culvert vacuum cleaning (downloaded from www.badgerinc.com/services/debris_rem.html) on June 17, 2010

Self-cleaning culvert design is a new approach in addressing the debris accumulation problem where the formation of sediment deposits is prevented using the hydraulic power of the stream. Muste et al. (2009) developed and tested a self-cleaning design method for a 3-box culvert configuration. Further research is focused on developing self-cleaning designs for 2-box culverts, conducting laboratory performance tests for culverts retrofitted with upstream and downstream cleaning fillets, assessing of the performance of self-cleaning culverts for overtopping design criteria, and estimating the effect of culvert modifications on the head losses through culverts.

3.1.14. ADDITIONAL MEASURES FOR IMPROVING HYDRAULIC PERFORMANCE

3.1.14.1 Roadway Overtopping and Flooding

If roadway overtopping frequency is greater than selected based on economics of the hydraulic design (see 2.2.6), the culvert may have insufficient design capacity for the projected peak flow conditions and may need to be re-designed.

Pelivanoski and Ivanoski (2010) described a case study of culvert redesign where seasonal storm events were repeatedly causing flooding. The box-culvert was originally designed to carry flows for the projected 100-yr peak flow conditions in the creek from one side of the road to the other without allowing the headwater depths to exceed acceptable levels, however. The culvert was also clogged with sand which further reduced the capacity of the culvert (Figure 233). Hydraulic analysis showed that even after the sediment removal (which would restore the original cross-section area of the culvert) the culvert would still not have the capacity to carry the anticipated volume of flood water. The solution was to open another culvert next to the current one.

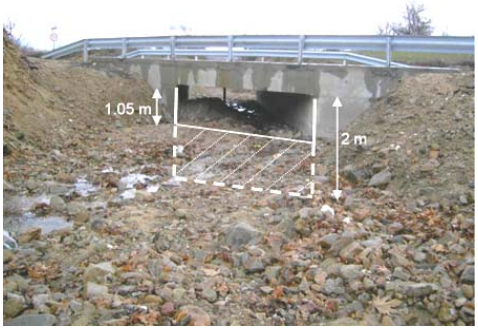


Figure 289. Reduction of the culvert height from original 6.5 ft to 3.5 ft due to debris (Pelivanoski and Ivanoski, 2010)

Modifying inlet geometry (improved inlets) can enhance the culvert hydraulic performance. Although the improved inlets are often considered more costly then installing a larger culvert, for some conditions they may offer a better option.

FHWA's *HEC No.13: Hydraulic Design of Improved Inlets for Culverts* **(**Harrison et al.., 1972) includes extensive research and test results for improving culvert flow capacity by modifying inlet geometry. The publication contains design examples and numerous charts and graphs. Improved inlets are bevels, side-tapers, and slope-tapers, which are modifications to the culvert entrance geometry. These improvements can greatly increase the performance of a culvert which is operating under inlet control. Design charts, tables and computation sheets are provided in the manual.

Grimaldi (1995) documented procedures to enhance culvert flow characteristics by making modifications to the inlet of culverts as a majority of culverts on Forest Service roads were found to flow under inlet control. Five different inlet treatments were considered: projecting inlet, mitered inlet, headwall and wingwall, beveled ring inlet, and side-tapered inlet.

Stormwater detention facilities (e.g., reservoirs, wet ponds) can be built immediately upstream of the existing culvert to produce a lower peak runoff rate at the culvert than it would occur without the facility. FHWA's HEC 22 (Brown et al., 2001) presented procedures for the design of stormwater detention facilities. FHWA's HDS No.5 (Norman et al., 2005) explained basics of storage routing, including a concept and the application to culvert design.

Channel aggradations (the vertical raising of the streambed over relatively long distances and time frames primarily due to sediment deposited from the streamflow) can contribute to an increased risk of flooding downstream of culverts. A manual by the U.S. Army Corps of Engineers (Watson et al., 1999) covered channel modification activities, including sediment control activities: sediment removal (dredging), the implementation of streambank and channel stability projects, better construction methods, trapping or storing sediments, structures for diverting flow, construction of sediment retention dams, and increased use of protective vegetation.

Some modifications (e.g., installation of baffles for fish passage that reduce the water velocity within and downstream of the culvert) can decrease the culvert's ability to convey water and increase the risk of flood damage above or below the site.

3.1.14.2 Damaged Embankments

Chen and Anderson (1986) discussed predominant modes of embankment failure during floods (the most common type of failure is caused by excessively high flood waters overtopping and eroding the embankment) evaluation effectiveness of some embankment protection measures (vegetated embankments, gabion mattresses, soil cement, geoweb, and enkmat). They developed a methodology to quantitatively determine embankment damage (a computer model to determine hydraulics of overtopping flow and associated erosion damage) and assess protective measures.

Channel degradation is a general and progressive (long-term), lowering of the channel bed due to erosion, over a relatively long channel length. Headcutting is channel degradation associated with abrupt changes in the bed elevation (headcut) that generally migrates in an upstream direction. In contrast, aggradation is the vertical raising of the streambed over relatively long distances and time frames, primarily due to sediment deposited from the streamflow. FHWA's HEC 20 (Lagasse et al., 2001) provided quantitative techniques for channel stability analysis, including degradation analysis, and introduced channel restoration concepts.

A manual by the U.S. Army Corps of Engineers (Watson et al., 1999) covered the selection and design of channel rehabilitation methods. The principle methods employed in channel rehabilitation projects for controlling erosion and sedimentation are:

- Bank stabilization, e.g., stone armor; flexible mattress made of concrete blocks, fabric and gabions, etc; dikes (a system of individual structures which protrude into the channel, generally transverse to the flow) and retards (a continuous structure approximately parallel to the streamflow), vegetative methods, etc.
- Grade control, e.g., dumping rock, concrete rubble, or some other locally available non-erodible material across the channel to form a hard point (structures often referred to as rock sills, or bed sills)
- Flow control, e.g., dams and reservoirs, and diversion canals.

3.1.14.3 Culvert Maintenance

Maintenance of culverts can be divided into routine, preventive and emergency maintenance. Ballinger and Drake (1995) outlined benefits of regular and preventive maintenance, discussed costs and legal implications to doing or not-doing maintenance work in a timely manner. Work options of these maintenance strategies are outlined in Table 35.

Table 35: Work options for culvert maintenance strategies (after Ballinger and Drake 1995, and Wyant, 2002)

Strategy	Objective	Work options
Routine Maintenance	To keep a culvert in a uniform and safe condition by repairing specific defects as they occur.	▪ Debris and sediment removal ▪ Thawing frozen culverts
Preventive Maintenance	Takes maximum advantage of the remaining unusable structure in a culvert to build a reconditioned culvert.	▪ Joint Sealing ▪ Concrete patching ▪ Mortar repair ▪ Invert paving ▪ Scour prevention ▪ Ditch cleaning and repair

FHWA's computer-based Culvert Management System (FHWA, 2001) is an automated tool to facilitate the coordination of culvert maintenance and replacement operations on a system wide basis. The software enables agencies to create an inventory of their culverts, assess them, and schedule repairs and replacements. It also helps agencies to develop maintenance plans and to estimate costs for installing, repairing, or replacing culverts (Schans, 2004).

3.1.15. SOIL IMPROVEMENT

3.1.15.1 Gravity Flow Grouting of Voids

This method is routinely used to fill voids in the soil under corroded or undermined inverts. The grout can be merely poured into the void from above (assuming it would completely fill the void without entrapping air) or be poured through a tremie pipe or tube (the grout fills the void from the bottom upward eliminating the potential problem of entrapped air). Portland cement based grouts and mortars, as well as chemical and foaming grouts (those that foam and expand when they come in contact with water) can be used (Ballinger and Drake, 1995).

3.1.15.2 Pressure Grouting of Voids

Pressure grouting is routinely used to fill voids in the soil behind the sides of culverts caused by piping

or exfiltration. Grouting is usually carried out from inside culvert: with two grout tubes in place (one directed to the bottom of the void and the other to the top of void), the grout is pumped through the bottom grout tube until it fills the void and starts to flow out through the upper tube. The voids can sometimes be filled from the roadway surface providing they can be accurately located. Portland cement based grouts and mortars, as well as chemical and foaming grouts can be used (Ballinger and Drake, 1995). Pumping shotcrete is an option to fill large voids under culverts. In Cincinnati, OH, a project was completed in 1999 in which a 25 ft long steel corrugated 96 in. diameter pipe was underpinned and all the voids around the pipe were filled through holes drilled through the sides and bottom of the culvert along the entire length of the pipe. On each end of the culvert, a Magnum Steel Push pier was used to help lift the sides and shotcrete was pumped into the holes (Figure 290). The slab on top was jacked: approximately twenty 2 in. holes were drilled through the top of the slab and pressure grouted thus raising the slab back to level (Chris Griggs and Dave Jacob, Dwyer Companies, personal communication).

Figure 290. Shotcrete pumping to fill a large void under the culvert (Dwyer Companies, 2009)

Figure 291. Shotcreting for stabilizing the slopes at the end of culvert (Dwyer Companies, 2009)

Figure 292. Restored culvert (Dwyer Companies, 2009)

3.1.15.3 Cement Based Compaction Grouting

Cement based compaction grouting is a method of soil stabilization where a low flow, cement based grout is injected into the soil to form a grout bulb. The grout does not permeate soil pores but displaces the soil while compacting it in the process. The method is used on settled roadways (Figure 293) and for remediation of sinkholes. A small diameter (2 in. to 4 in.) steel casing is advanced through the zone to be improved, and a stiff mortar-like grout is injected at high pressure.

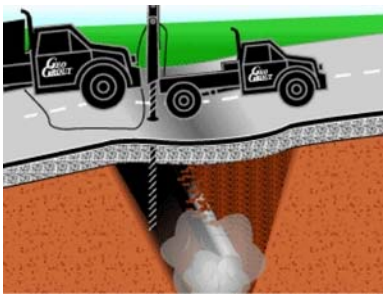


Figure 293. Compaction grouting used for soil stabilization in the trench (http://www.geogrout.com/soil.shtml)

In 1996/97, Caltrans used compaction grouting to stabilize the soils surrounding storm drains in Los Angeles, CA where major sinkholes occurred due to infiltration of soil into the storm drain through insufficiently sealed pipe joints. Storm-drains were reinforced concrete (RCP) and corrugated metal (CMP) pipes, total length 14,460 ft, and ranging in diameter from 24 in. to 54 in. The project involved approximately 6,500 cubic yards compaction grouting (Caltrans, 2003).

In 2007, Florida DOT used compaction grouting to stabilize the soil around a corrugated metal culvert that was re-rounded and relined after being badly damaged by an HDD contractor, but exterior holes in the soil still remained. A total of 810 lb of flowable fill concrete (made of weak Portland cement and fly ash) was injected from the road surface. Six 2 in. holes were drilled through 18 in. of asphalt and a layer of compaction, and 1.25 in. pipes were pushed into the holes. The holes were filled with grout using a grout pump to within 1 in. of the roadbed, and the remaining depressions were plugged with asphalt (Dayton, 2007).

3.1.15.4 Polymer Compaction Grouting

In this method of soil stabilization, high-density hydro-insensitive polymer is injected into the soil where it expands from its original liquid volume reaching its strength within a short time to densify and stabilize low-density compressible soils, and fill any voids. To make injections, multiple holes (0.5 in. to 0.75 in. in diameter) are drilled to the various effected strata levels using specific injection grid patterns engineered to maximize the support. The method can be applied from the culvert structure (Figure 294) or from the surface to a depth up to 30 ft (Uretek USA, 2005; Uretek USA, 2008b).

Figure 294. Injecting polymer material through holes drilled in the base of the culvert (Uretek USA, 2008a)

New Mexico DOT used polymer compaction grouting to fix stability issues with a corrugated metal culvert under a highway in 2002. The 155 ft long multiplate culvert, 14 ft tall and 21 ft wide, was constructed 22 ft below the road surface that had settled as much as 6 in. Penetrometer tests revealed strong base and sub-base soils above the crown of the culvert, and poor soil compaction and large, deep voids in several key weight-bearing locations around the culvert itself. For application, small 0.625 in. holes were drilled through the corrugated pipe using a radial pattern. The densified soil was pushed upward restoring the road to profile (Roads and Bridges, 2008).

Alabama DOT addressed a soil settlement problem with a 3-barrel culvert underneath a busy divided highway in Alabama in August 2002. The polymer material was injected using a multi-point pattern spread evenly across each of the culvert barrels. The two barrels directly above the deepest section of the soil void accepted the largest amount of grout (Uretek USA, 2008a).

3.1.16. PIPE REPLACEMENT ON NEW ALIGNMENT

Replacement on new alignment basically involves new culvert installations and is not a subject of this study, so it is only briefly covered in this section. When a new culvert replacing a deteriorated or hydraulically inadequate culvert is installed on a new alignment, the existing culvert can either be abandoned or repaired to remain in service. An abandoned culvert should be filled with concrete or another structural fill to ensure stability of the road surface in the face of continuing culvert deterioration.

Knoyle (2007) described a case study of replacement with new concrete culvert (Hwy 274 Culvert Replacement in Ottawa, ON, Canada). A corrugated metal culvert, 265 ft long and 60 in. in diameter, which consisted of two sections (190 ft and 75 ft, laid at 15° horizontal angle) was abandoned and a concrete culvert 70 in. in diameter was installed with jack and bore tunneling. Stearable shielded TBM was used.

Staheli et al. (1998) provided guidelines for installation of pipelines beneath levees using directional drilling. Notable guidelines for HDD installations are, for example, CALTRANS (2003) and HDD Good Practices Guidelines (Bennett et al., 2008).

One important issue associated with trenchless new installations is settlements that generally can be characterized as either large settlements or systematic settlements. Large settlements occur primarily as a result of loss of ground due to over-excavation and can lead to creation of voids or sinkholes above the bore. This risk is minimized through a comprehensive geotechnical investigation, selection of proper means and methods, use of ground improvement measures and good workmanship by the contractor. Systematic settlements associated with trenchless construction are primarily caused by the collapse of the overcut or annular space between the new pipe and excavation, and to a lesser extent by elastic deformations of the soil ahead of the advancing bore. Systematic settlements can be controlled by selecting an appropriate depth for the installation, maintaining a reasonable radial overcut, keeping the annulus filled with drilling fluids and by grouting the annulus after pipe installation (Bennett, 2009).

3.2. REHABILITATION METHODS DESIGN

3.2.1. INTRODUCTION

This chapter first examines lining classification and different design limit states requiring consideration. Next, the literature dealing with design calculations for resistance to external fluid pressure is discussed, followed by consideration of design for resistance to external earth and vehicle loads. These discussions relate to design of slip-liners, cast in place pipe liners, grouted and non-grouted spiral wound liners, and other products where the liner is required to resist external water or earth loading. This is followed by consideration of sprayed on linings, spot repairs, pipe bursting, and soil stabilization. Lastly, considerations of hydraulic design for liner systems are examined.

3.2.2. LINING CLASSIFICATION AND DESIGN LIMIT STATES

The design method used for repair using a liner is dependent on the nature of the interactions between the liner and the existing structure. A number of frameworks have been developed to address this characteristic of liner performance.

Design practice commonly used for cured-in-place pipe and other liner systems in North America is described in ASTM F1216-07b. Design is based on an assessment of the condition of the damaged rigid pipe. If the host pipe is characterized as “partially deteriorated”, it is treated as hydraulically flawed, but still able to support soil pressures throughout the design life of the rehabilitated pipe. The liner is therefore designed to support only the external hydraulic loads due to groundwater. If it is characterized as “fully deteriorated” it is also considered to be structurally flawed, so that design for earth loads is required. Unfortunately, the classification system relies on a simple assessment of the level of out-of-roundness in the damaged pipe (the damaged rigid pipe is classified as “fully deteriorated” if its vertical pipe diameter has decreased more than 5%.). This is not a rational assessment of the remaining structural capacity of the pipe being repaired or the soil surrounding it (as discussed in as discussed subsequently in section 3.2.4).

Another example is the procedure outlined by WRc (2001) which classifies repairs into two categories:

- Type 1: are systems where the liner, the existing structure, and grout if it is used, adhere to each other so that composite behavior results.
- Type 2: are systems where the liner acts as an independent structure, and the liner does not adhere to the pipe being repaired or the grout if it is placed in between.

WRc (2001) design of a type 1 liner is based on creating a rigid composite pipe to carry all earth and live loads. Design of only one liner class (GRP pipe) considers long term buckling resistance of the liner to external water pressure, though others consider short term buckling resistance to grout pressure.

WRc (2001) design of a type 2 liner is based on the existing deteriorated structure and the soil surrounding it being in a stable state under the action of external earth loads. The primary purpose of the type 2 liner is therefore to restore the hydraulic integrity of the system. As a result, the liner design only considers the ability of the liner to resist the external fluid pressures. This type 2 classification is similar, therefore, to the 'partially deteriorated' designation used in ASTM F1216.

WRc (2001) also includes consideration of loads associated with floatation forces during design of grouted Type 1 and Type 2 liners.

Another example framework is that set out in EN 13689 which characterizes various aspects of the pipe liner system, both for gravity flow and pressure pipes. After listing repair techniques (a list like that provided earlier in this review), it goes on to discuss design for each of these liner systems. The standard suggests that the following issues need to be considered during design of liners within gravity flow pipes:

A. Installation loads
   1) Lining pipe preparation forces (e.g. Section reduction, spiral winding);
   2) Insertion forces (tensile, compressive, bending, torsion);
   3) Reversion forces (pressure, thermal);
   4) Grouting forces (external pressure, flotation);
   5) Residual effects of the above installation forces in the permanent works.
B. Internal loads:
   1) Surcharge pressure;
   2) Thermal loads due to temperature of transported fluid.
C. External loads:
   1) Transferred soil loads, from overburden soil weight, traffic surcharge etc.;
   2) Ground movements, from differential settlement, frost action, earthquakes etc.;
   3) Point loads from irregularities of the existing pipeline;
   4) Thermal loads due to the environment;
   5) Groundwater pressure, and/or negative pressure (vacuum).

The German framework ATV M127 is explained by Falter (2001). It includes classification of the deteriorated structure into three categories:

I. Host pipe structure is considered safe, but leaking; no cracks exist except those resulting from shrinkage
II. Host pipe-soil system is considered safe; however four longitudinal cracks exist giving a deformation mechanism, though with deformations (change in diameter) less than 5 % of diameter
III. Host pipe-soil system not considered safe for long-term conditions; there are four longitudinal cracks, and deformations bigger than 5 % of diameter

Treatment is not clear for flexible pipes as well as rigid pipes between I and II (those with less than four longitudinal fractures). Design for the following limit states is examined in detail in subsequent sections:

i. Design for external fluid pressures, Figure 295a (e.g. considered for WRc Type 2 liners; ASTM F1216 design; EN 13689 items A4, C5).
ii. Design for external earth and live loads, Figure 295b (e.g. considered for WRc Type 1 liners; ASTM F1216 'fully deteriorated' pipes; EN 13689 item C1).
iii. Design for floatation force (EN 13689 item A4)

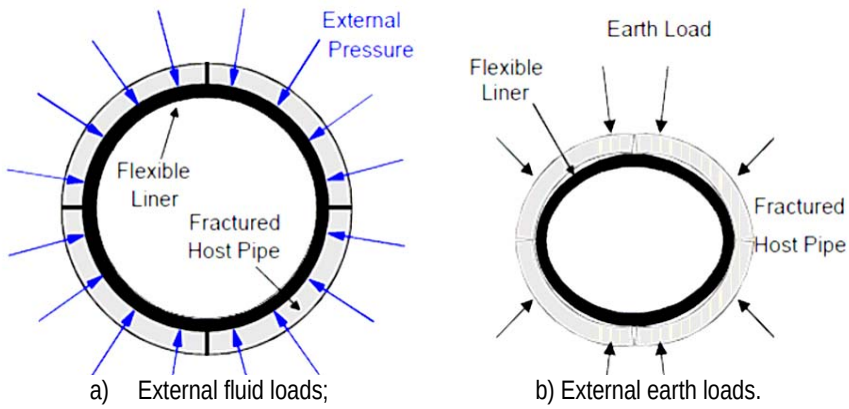


Figure 295. Two loading conditions requiring consideration (Moore, 2005)

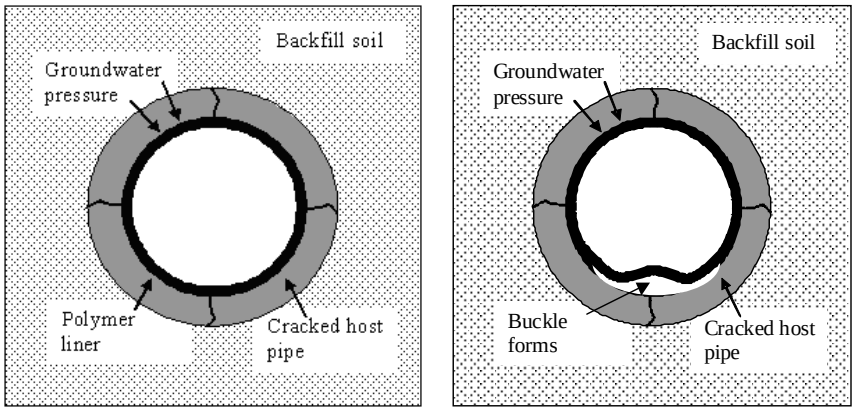
3.2.3. LINING DESIGN FOR EXTERNAL FLUID PRESSURE

3.2.3.1 Introduction

From the outset during development of cast in place sewer liners in the UK (Aggarwal and Cooper, 1984), it was understood that liners needed to resist the effect of external fluid loading:

- For sewers, these external fluid loads are commonly applied by the groundwater acting through joints, factures, corrosion voids or other means of passing through the old structure to be repaired (Moore, 1998).
- For culverts, this external fluid load may arise after flood events, since the groundwater level in granular backfill will rise during storm events, and may remain perched above the culvert after the stormwater level inside the culvert structure has subsided (Moore, 2008).
- Some repair processes involve use of grout between the liner and the old structure, and the liner needs to be able to withstand the pressure of that grout without buckling (Moore, 2005).

Figure 296 shows one type of lined pipe system being considered. The external water or grout pressures acting on the polymer liner lead to compressive ring thrusts that can cause the liner to buckle at the invert (the location of maximum pressure), at the location of maximum local radius (e.g the sidewalls of an egg-shaped sewer), or at a local imperfection, Figure 296b. Figure 297 shows a buckle observed during tests on a plain HDPE pipe under external pressure (Santamaria, 2003).



a. Water pressure through fractures b. Buckle at invert

Figure 296. Cause and outcome of fluid pressure buckling (Moore, 2005)

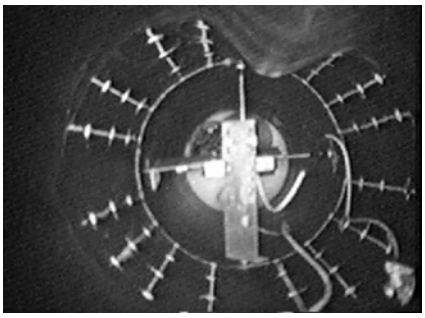


Figure 297. Buckle observed during a laboratory test (Santamaria, 2003)

Levy (1884) determined the external fluid pressure that can be supported by an unrestrained cylindrical shell of modulus *E*, Poisson's ratio ν, thickness *t* (or second moment of area *I*) and radius *R*:

$$P_{cr} = 0.25\,[E/(1-\nu^2)]\,(t/R)^3 = 3[E/(1-\nu^2)]I/R^3$$

When an unrestrained shell buckles, it adopts an elliptical (i.e. oval shaped) deflection pattern (Moore, 1989).

However, liner stability (i.e. buckling strength) is significantly enhanced by the external restraint provided by the old culvert pipe surrounding the liner (Aggarwal and Cooper, 1984), though that support is compromised by imperfections in the liner (e.g. El Sawy and Moore, 1997; Moore, 1998).

3.2.3.2 Experimental Investigations

When Insituform developed the original process of relining sewers, they recognized that the Levy solution would produce very conservative designs, since the host pipe surrounding the liner provides radial restraint against the elliptical buckling mechanism. Insituform funded the tests of Aggarwal and Cooper (1984), and developed a design approach where an experimental (empirical) "enhancement factor" K is used to scale up the Levy solution to account for the external restraint

$$P_{cr} = K\ 0.25\,[E/(1-\nu^2)]\,(t/R)^3 = 3K[E/(1-\nu^2)]I/R^3$$

Testing on the Insituform product by Aggarwal and Cooper (1984) implied that *K* is 7, and this value is often quoted in design codes (e.g. the practice for cured-in-place pipe liner systems described in ASTM F1216-07b).

However, a thin shell with external restraint is subject to non-linear bucking, Moore and El Sawy (1996), El Sawy and Moore (1997), Moore (1998). While linear buckling phenomena are controlled by the undistorted geometry of the system (e.g. Euler buckling of a column, where stability can be considered in the straight member without consideration of imperfect geometry), non-linear buckling phenomena are significantly influenced by the local geometry of the structure, making them 'imperfection sensitive'. El Sawy and Moore (1997; 1998) and Moore (1998; 2005) explain that the empirical correction factor K varies as follows:

- K < 7 when the liner is thick, or when there are significant geometrical imperfections
- K > 7 when the liner is thin and relatively free of defects.

Moore (1998) summarized the testing work performed between 1984 and 1995, as shown in Table 1. This work consisted of:

- Tests reported by Aggarwal and Cooper (1984) as discussed earlier
- Tests conducted at the Trenchless Technology Center, e.g., Guice et al. (1994)
- Tests reported by Bakeer and Barber (1995)
- Tests reported by Lo and Zhang (1994)
- Tests conducted at the University of Bradford under the supervision of Boot (those of Welch, 1989; and subsequent experiments by Omara, 1997).

Table 36. Buckling tests performed on polymer liners (as summarized by Moore, 1998).

Source	Diameter & length	Liner type	DR	Pressure source	Host pipe	Fit	# of tests	Ends
Aggarwal and Cooper (1984)	25 cm L/D=4	RIF CIP	30 to 90	Water horizontal	Steel cylinder	Loose	49	Clamped
Baker & Barber (1995)	15.20 cm L/D=10	PE SL	26 to 32.5	Water horizontal	Steel cylinder	Loose		Clamped
Guice et al. (1994)	30 cm L/D=6	RIF CIP PVC SL	31 to 61	Water horizontal	Steel cylinder	Loose	200	Clamped
Lo and Zhang (1994)	15 cm	RIF CIP	54 to 55	Water horizontal	Steel cylinder	Loose	12	Clamped
Welch (1989)	45 cm L/D=21	RIF SL	45	Air vertical	Clay pipe segments	“Tight”	14	Free

Moore (1998) discussed these tests in detail, in particular focusing on the effect of the boundary conditions at the ends of the liner samples. He concluded that the tests at Bradford with ‘free’ ends were the best representation of ‘long liner’ responding under plane strain conditions. Subsequent analysis by Hall and Zhu (2000) indicates that end effects for the TTC tests were modest. Gumbel (2001) provides further discussion. Comparisons between experimental measurements and theoretical calculations are presented in the next subsection.

3.2.3.3 Theoretical Investigations

3.2.3.3_1 Nonlinear Buckling Solution for Perfect Circular Liners, Glock (1977)

Glock (1977) analyzed liner buckling under external pressures and developed an approximate non-linear buckling solution. The critical buckling pressure for a liner encased in a perfect circular cavity subjected to external loads (P_{cr}) is given as:

$$P_{cr} = 1.0025 \frac{E}{1-\nu^2}\left(\frac{t}{D}\right)^{\frac{11}{5}}$$

where E is the liner's modulus of elasticity, v is the Poisson’s ratio, t is the liner thickness, and D is the internal diameter of the rigid host pipe. This solution follows the experimental data of Aggarwal and Cooper (1984) more effectively than the ASTM design equation, . Furthermore, it does not rely on the empirical enhancement factor K, which has only been established for cast in place liners like those manufactured by Insituform.

The ATV M127 procedure for hydrostatic buckling of the liner is based on the Glock solution (Falter, 2001).

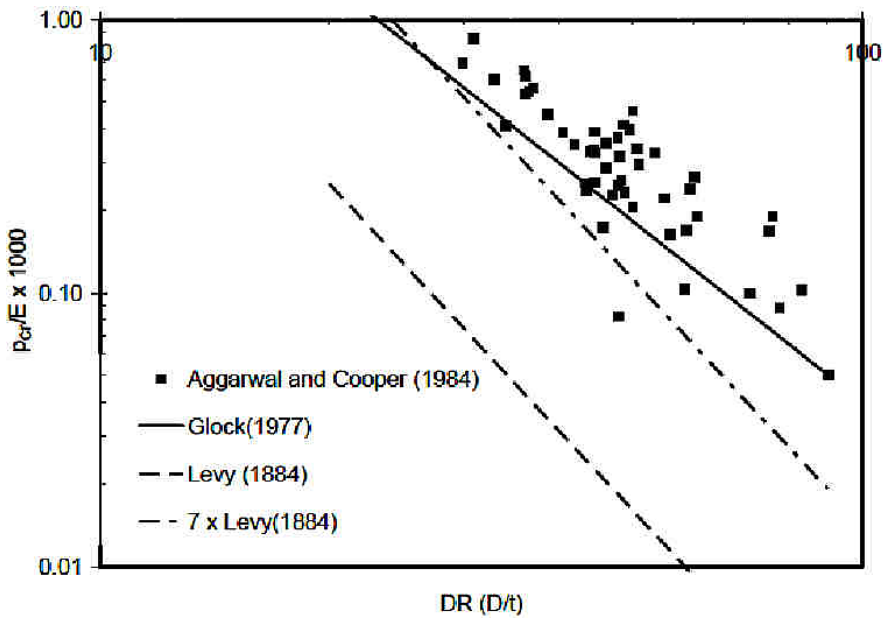


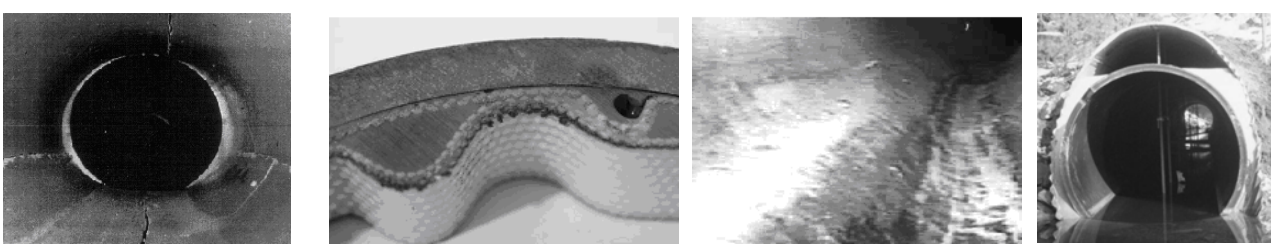
Figure 298. Experimental data, unfactored and factored Levy theory, and the nonlinear restrained shell buckling theory of Glock (1977) reported by Moore (2005).

3.2.3.3_2 Buckling Solution for Imperfect Liners, El Sawy and Moore (1997)

Moore and El Sawy (1996) and El Sawy and Moore (1997; 1998) classified the different kinds of imperfections that influence liner stability (Figure 299) idealizing them in three ways (Figure 299):

I. Elliptical geometry (Figure 299a, Figure 300a, Figure 100d)
II. Longitudinal imperfections - wavy or flat (Figure 299b, Figure 300b, Figure 100e)
III. Initial gap between the liner and the old culvert (Figure 299c, Figure 300c)

These imperfections are either associated with the damaged structure being repaired, or are characteristics of the liner system being used (see Law and Moore, 2007a, and subsequent sections).

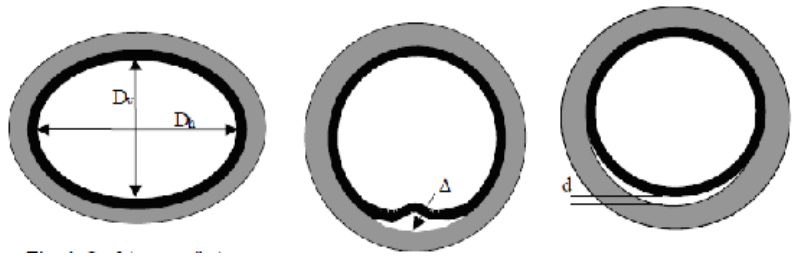


a. Ovality (Law and Moore, 2007b)

b. Local waviness in 2 cast in place liners (Ampiah et al., 2008; Ian Moore, Queen's University, personal communication)

c. Gap between liner and culvert being repaired (Ian Moore, Queen's University, personal communication)

Figure 299. Imperfections that influence buckling strength under external fluid pressure.



a. Ovality b. Wavy imperfection c. Gap

Figure 300. Idealized imperfection geometries (Moore, 2005; Gumbel, 2001)

Moore and El Sawy (1996) quantified how small wavy imperfections in tight fitting pipe liners influence stability, and developed 'lower-bound' design envelopes for imperfect liners. They concluded that thin liners are very sensitive to imperfections, while thicker liner systems are less vulnerable to stability loss due to local imperfections. A parametric study to examine liner stability in elliptical host pipes was reported by El Sawy and Moore (1997). Modified versions of Glock's equation have been developed to account for local imperfections and elliptical geometry, Moore (1998).

a) For tight fitting linings.

$$P_{cr} = \frac{E}{1-\nu^2}\left(\frac{t}{D}\right)^{\frac{11}{5}} e^{-0.56\Delta/t} e^{-q/18}$$

where $R_\Delta = e^{-0.56\,\Delta/t}$ is a correction factor to account for a small wavy imperfection of radial amplitude Δ, Figure 300b; and $R_q = e^{-q/18}$ is used to quantify the effect of percentage ovality q, Figure 300a, where q is defined using pipe 'rise' D_v and 'span' D_h (Figure 300a):

The first of these terms reveals the substantial decreases in stability that result when the liner has initial

$$q = 100\frac{D_h - D_v}{D_h + D_v}$$

wavy imperfections. An amplitude Δ of just one liner thickness reduces bucking strength by a factor of nearly two.

b) For loose fitting linings.

$$P_{cr} = \frac{E}{(1-\nu^2)}\left(\frac{t}{D}\right)^{11} R_q R_d \geq \frac{2Et^3}{(1-\nu^2)D^3}$$

where

$$R_d = \frac{1 + 46\left(\frac{t}{R}\right) + 380\left(\frac{t}{R}\right)^2 + 500\left(\frac{d}{R}\right)\left(\frac{t}{R}\right)}{1 + 46\left(\frac{t}{R}\right) + 380\left(\frac{t}{R}\right)^2 + 100\left(\frac{d}{R}\right) + 1.5\left(\frac{d}{t}\right)}$$

is a function of the initial gap d between the liner and the host pipe (the difference in the internal diameter of the host pipe and the external diameter of the liner, Figure 300c). This factor is described and quantified by El Sawy and Moore (1998). The minimum buckling strength shown above corresponds to a cylindrical shell placed directly inside a fluid.

Moore (2005) used the experimental data of Welch (1989) and Omara (1997) to examine whether buckling can be successfully captured using the nonlinear model of Glock (1977) modified by the ovality correction factor, Figure 301. The correction factor R_q based on the finite element results sits above the data points, whereas calculation of ovality correction together with wavy imperfection where the liner straddles the longitudinal fracture at the pipe invert $R_q^* = R_q\, R_\Delta$ lies close to the mean of the experimental measurements (an expression for Δ as a function of q for this case is given by Moore, 2008).

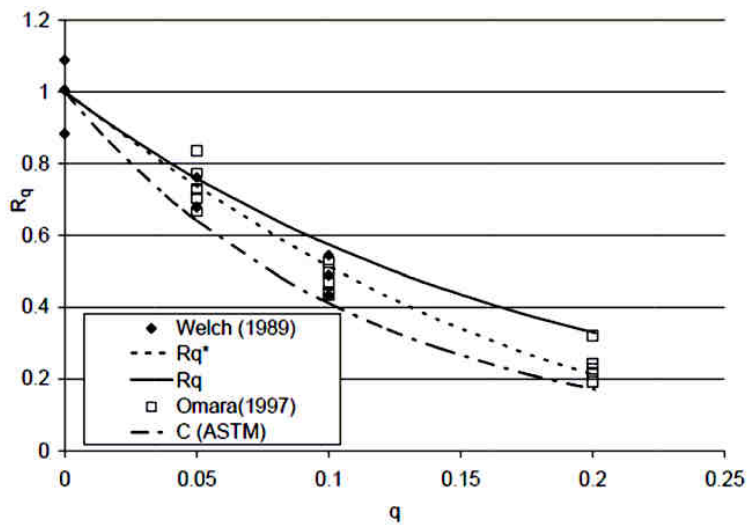


Figure 301. Comparison by Moore (2005) of the buckling theory for elliptical structures (El Sawy and Moore, 1997; ASTM F1216) against the test measurements of Welch (1989) and Omara (1997).

3.2.3.3_3 Imperfect Liner Solutions of Boot

Boot at the University of Bradford has also contributed to the consideration of liner buckling, using semi-analytical and finite element solutions to examine imperfect liners (Boot 1998, Javadi and Boot 1998, Boot et al. 2001). Boot's approach has been to develop computer programs based on approximate semi-analytic solutions to calculate the correction factors, rather than simplified design equations of Moore and El Sawy (1996) and El Sawy and Moore (1997; 1998) and Moore (2005) based on finite element results.

3.2.3.3_4 Liner Analysis of Falter

Falter (1996; 2001; 2004) has developed structural analyses to examine liner buckling considering a range initial geometries, and imperfections. This work is incorporated into German design practice through ATV M127-2. It involves use of proprietary finite element software for each liner design. Glock-based design is used. However, it appears that very conservative theoretical assumptions are made about levels of imperfection which eliminate much of the potential economic benefit of using a design procedure that accounts for the experimental evidence.

3.2.3.3_5 Noncircular Liner Performance of Théopot

The Glock theory has been extended to predict the response of restrained non-circular liners to external hydrostatic pressure (Thépot, 2000). For many shapes, including the standard 3 x 2 egg over a defined range of thicknesses (equivalent SDRs), buckling proves to be critical.

3.2.3.3_6 Other Studies

Zhao and Hall (2004) have explicitly examined the impact of installing a cast in place liner with the inner surface corrugated and the outer surface against the host pipe smooth (though it is not clear how this would come about). However, it provides some guidance on the performance of a polymer liner installed within a corrugated metal culvert (say, where outer wall is corrugated and inner wall is smooth).

Boot and Welch (1996), Nassar and Yousef (2002) and Zhao et al. (2001) have studied creep (time dependent) buckling of polymer liners, both with experiments and analysis. These studies can provide guidance on whether a simple approach can be employed whereby long term 'secant' modulus for the polymer is used in the buckling design models to ensure liner stability over the required service life of the repair.

3.2.3.4 ASTM Design Equation for Fluid Pressure Buckling

ASTM F1216-07b provides "non-mandatory" appendices that outline calculation procedures for liner design for a number of limit states. The thickness required to provide adequate resistance to external fluid pressure is determined from the original semi-empirical buckling equations developed by Insituform:

$$P = \frac{2KE_L}{(1-\nu^2)} \cdot \frac{1}{(SDR-1)^3} \cdot \frac{C}{N}$$

where hydraulic load P is defined as a function of enhancement factor K, the long-term modulus of elasticity of the liner E_L, the standard dimension ratio of the liner SDR, ovality reduction factor C, and the factor of safety N (generally 2.0). Ovality factor used in the ASTM procedure is based on the effect on the Levy solution of an elliptical shell geometry.

$$C = [(1;q) / (1 + q)^2]^3$$

This appendix should not be used for design of liners in odd-shaped structures (e.g. egg shaped sewers or pipe arch culverts) since the radius of curvature of the flattest sections of the structure is greater than that of the ellipse for which ovality correction is intended. Much higher equivalent "ovality" must then be employed.

While the standard specifies that K=7 only for some cases, and that designers should consult the liner manufacturer, the procedure is almost invariably employed using this value. As discussed earlier in relation to , this produces a result that does not match the experimental evidence as effectively as the nonlinear buckling models.

3.2.3.5 PPI Design for Fluid Pressure Buckling

PPI (2006b) has recommended design procedure for sliplining non-pressure HDPE as follows:

- Select a slip-liner diameter. Use the largest possible size based on the size and condition of the existing conduit.
- Assume a thickness for the pipe liner and compute the critical buckling pressure for the unsupported pipe (the Levy solution, also often referred to as the Love equation). Compare the buckling load to the maximum anticipated external hydrostatic pressure. Revise the assumed wall thickness until a factor of safety of at least 2 is obtained. (A more detailed evaluation is needed if the pipe will be subjected to internal pressure.)

This procedure therefore ignores any potential benefit from the external support provided by the pipe being repaired. It should generally be very conservative.

3.2.4. LINING DESIGN FOR EXTERNAL EARTH AND LIVE LOADS

3.2.4.1 Overview

The effect of earth loads on liners has been the subject of considerable discussion in the community of researchers, designers and product suppliers (e.g. Schrock and Gumbel, 1997; Gumbel, 2001; Moore, 2005). Initially, the liner was designed as a pipe sitting directly within the soil, and the design objective was to ensure the liner could resist buckling under those external earth pressures. More recently, local bending stresses have been investigated under the external earth loads, Law and Moore (2003b; 2007b).

The effect of earth loads on the liner will be different for grouted and non grouted systems:

a. Ungrouted systems feature direct contact between the liner and the damaged pipe being repaired.

b. Grouted systems produce a composite structure that means that the liner interacts with the grout, and the grout with the existing pipe. Unless damage in the existing pipe (e.g. fractures in rigid pipes) leads to similar damage in the grout after installation (e.g. fractures with locations and orientations the same as the longitudinal or circumferential fractures in the host pipe), then the loads from the surrounding ground and the vehicles may not reach the liner at all.

Much debate has resulted as researchers and engineers in industry have considered whether any liner placed within the deformed pipe is required to carry earth earth loads (e.g. Schrock and Gumbel, 1997). It is now generally accepted that the liner will not experience the result of earth loads (Law and Moore, 2003b, 2007b) unless:

- Additional overburden soil is added at the site after the liner is installed (say as a result of elevation of the road embankment or placement of additional pavement over the repaired pipe)
- The sewer pipe or culvert is close to the ground surface and subjected to the influence of vehicle load
- Excavation and backfilling of other utilities takes place in the vicinity of the repaired pipe, leading to temporary and perhaps permanent changes in ground stress
- Further segments of relined host pipe lose structural capacity after the liner is installed (as a result of further deterioration in a reinforced concrete sewer pipe, for example, or as a result of loss of ground support to a clay pipe).

Pipe deformations under the influence of changes in earth loads need to be calculated and evaluated if any of these eventualities are expected. Furthermore, live loads (vehicles at the ground surface) need to be considered, particularly for culverts at shallow burial.

3.2.4.2 Liner Resistance to External Earth Loads

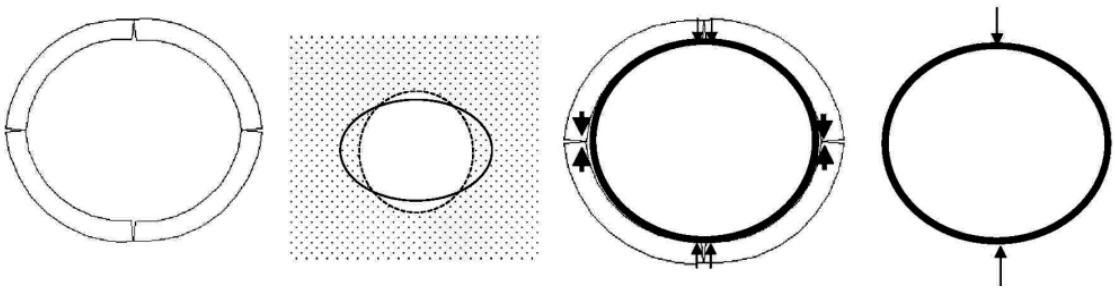
3.2.4.2_1 Ungrouted Repair of Damaged Rigid Pipe

3.2.4.2_1a Introduction

The nature and potential importance of the static pipe-liner response has been discussed by Moore (1998). If the pipe being repaired is seriously damaged, it will lose its structural integrity and the segments of original pipe can form a failure mechanism, Law and Moore (2003a). A rigid pipe with

significant wall loss will have reduced moment capacity. Alternatively (and likely more commonly), erosion of the backfill soil surrounding the pipe will reduce the effective 'bedding factor' (or increase moment as a result of lateral soil support) so that bending moments increase, Tan and Moore (2007). Both these alternatives result in overload fractures at the crown, invert, and springlines, Figure 302a. This permits ovaling deflections, Law and Moore (2003a, b), Spasjovic et al. (2004; 2007). The vertical stresses in the ground generally exceed the lateral stresses so vertical diameter decreases and horizontal diameter increases. These deformations essentially redistribute any non-uniform earth pressures into the surrounding ground (similar to the effect of the ovaling deflections seen in flexible pipes, Law and Moore, 2003b, Figure 302b).

The appendix in ASTM F1216-07b uses 5% deformation in rigid pipe as the indicator that the liner must be designed to carry the earth and live loads (the 'fully deteriorated' case). However, once the four fractures occur, the liner can experience the influence of external earth loads, even at 1% change in pipe diameter. Therefore, this classification approach should be discarded.



a. Four hinge segments. b. Flexible pipe deformations c. Local bending at crown and invert d. Parallel plate loading approximation

Figure 302 Fractured pipe kinematics and statics under earth loads (Law and Moore, 2003b)

3.2.4.2_1b Liner-Host Pipe Interaction and Local Bending in the Liner

One implication of the movements of the segments of old pipe shown in Figure 302a is local bending in any liner installed within, Figure 302c. Local stress and strain where the liner drapes over the parting corners of the segments of host pipe at the crown and invert is a particular concern. Law and Moore (2003b) capture this local bending using a parallel plate loading analogy, Figure 302d, based on their laboratory tests (Figure 303). They found that earth load thrusts are primarily carried by the old fractured pipe, with most of the overburden load passed between the upper and lower segments at the springline contact points. Therefore:

- Bending at the crown and invert needs to be assessed.
- Hoop thrusts do not need to be considered.

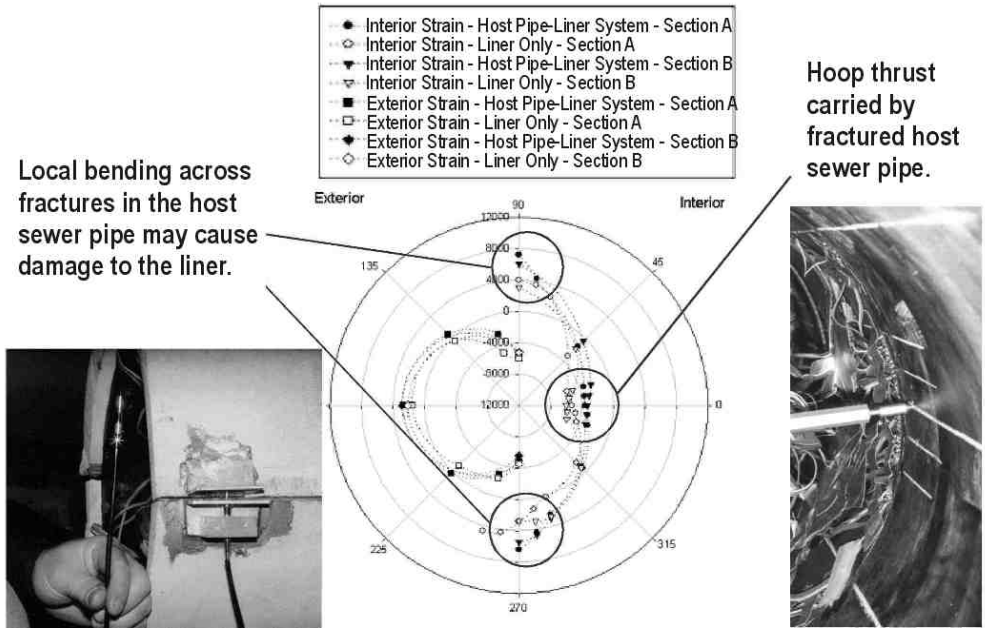


Figure 303. Lined pipe specimen tested under earth loading (Law and Moore, 2003b)

3.2.4.2_1c Calculation of Change in Pipe Diameter

Those tests have also demonstrated that a pipe that fractures after pipe burial and relining (when the overburden pressure is acting on the old sewer before it fractures) will experience the effect of the full weight of soil over the pipe, or

$$\sigma_v = \gamma h$$

for unit weight of soil γ and burial depth h. The value of K_0 for most soils can be estimated from the friction angle of the material ϕ: $K_0 = 1\text{-}\sin\phi$.

Moore (2008), Tan and Moore (2007) and Tan (2007) provide guidance on methods of calculating the deflection of fractured rigid pipe culverts. Finite element analysis clearly indicates that the change in horizontal diameter ΔD_H can be estimated using flexible pipe theory, Figure 304. Spangler's equation could be employed, or one of the elastic continuum solutions (e.g., Moore, 2001):

$$\Delta D_H = -4\,\sigma_v\, OD_{pipe}\,(1\text{-}K_0)\,(1\text{-}\nu_s)\,(1+\nu_s) / [(3\text{-}2\nu_s)\,E_s]$$

for soil modulus E_s, Poisson's ratio ν_s, coefficient of lateral earth pressure K_0, change in vertical earth pressure σ_v, and outside diameter of OD_{pipe}.The kinematics of pipe deformation provides a relationship between changes in vertical and ΔD_V and horizontal diameters

$$\Delta D_V = -\Delta D_H\left(1 - \frac{2.t_{pipe}}{OD_{pipe}}\right)$$

where the rigid pipe culvert has wall thickness given by t_{pipe}.

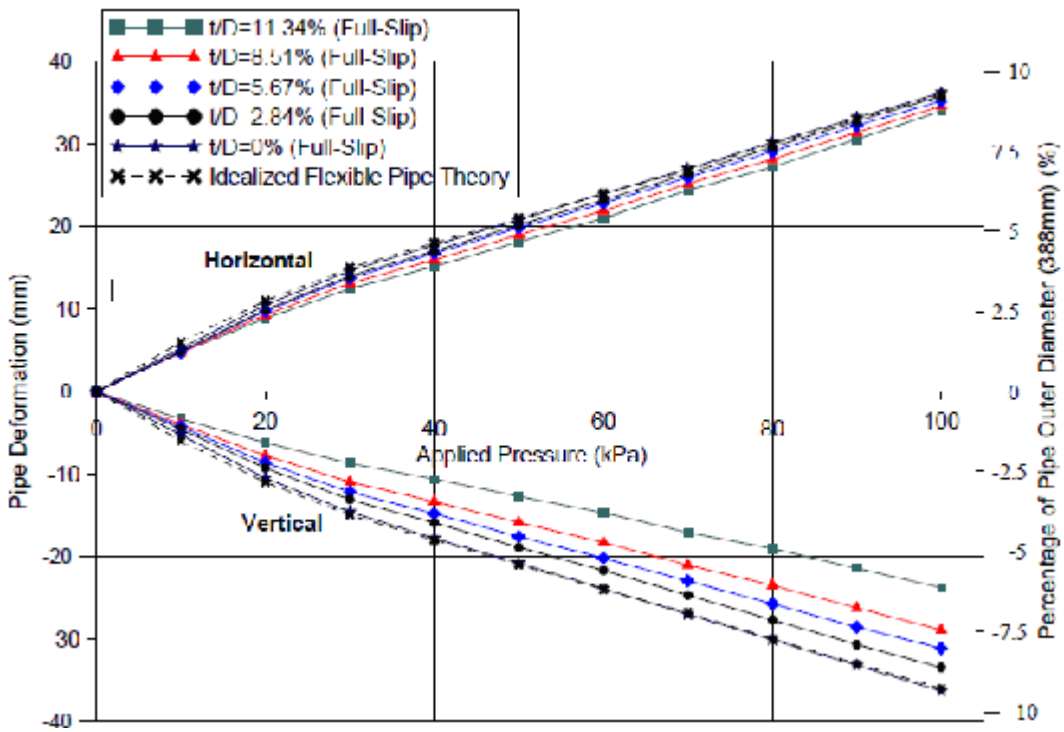


Figure 304. Finite element calculations for fractured rigid pipe deflection versus flexible pipe theory, Moore (2008).

3.2.4.2_1d Calculation of Local Bending Strain

A liner placed within a fractured culvert can experience local bending at the crown and invert, where the fragments of old pipe come into direct contact with the liner, Figure 302c. Using the approximation developed by Law and Moore (2003b) based on a ring under parallel plate loading, Figure 302d, local bending strain can be estimated as

$\varepsilon_{cr} = \varepsilon_{in} = 3.9 \ \sigma_v R_{host} c/[R_{liner}^2 E_s]$

for vertical earth pressure σ_v and typical values of $\nu_s = 0.3$ and $K_0 = 0.4$.

If pipe inspection provides an estimate of the change in pipe diameter ΔD_v, strain can be calculated from that observed deflection as:

$$\varepsilon_{cr} = \varepsilon_{in} = \pm \frac{\Delta D_v c}{2\pi \overline{R}_{liner}^2 \left(\frac{\pi}{8} - \frac{1}{\pi}\right)} = \pm \frac{2.139 \Delta D_v c}{\overline{R}_{liner}^2}$$

This performance limit sets an upper bound on the thickness of the liner that can be used in a given application, whereas the fluid pressure buckling limit state sets a lower bound thickness. Acceptable liner solutions will lie between these.

Finite element analyses are also available to calculate local stresses, like those reported by Falter (2001) and Law and Moore (2007b).

3.2.4.2_1e Effect of Soil Voids or Other Deterioration in the Backfill Soil

This implicitly includes any effect of soil void or other deterioration in ground stiffness (see discussion later in this subsection) so is likely preferable to the preceding equation which needs the user to estimate soil modulus. In many cases, the effective soil modulus available to resist ovaling deformations will be significantly less than it was when the backfill was first placed (field cases indicate the observed deflections in rigid pipe are often much greater than those that would be calculated using the design soil modulus, not surprising since it is likely that soil deterioration that caused the pipe to fracture in the first place, Moore, 2008).

Value c is the distance from the mid-surface of the liner to its extreme fibers, equal to half the wall thickness for plain pipe. This expression uses two different pipe radii, namely the external radius of the host pipe R_{host} and the mean or average radius of the liner, R_{liner}. These strain values should be compared with the long term strain capacity of the liner material (a typical strain limit for HDPE is 5%, for PVC is 5%, and for the cast-in-place polymer used by Insituform is 1%).

3.2.4.2_1f Soil voids and backfill deterioration

Centrifuge model tests at the University of Cambridge (Spasojevic et al. 2004; 2007) have examined how a lined sewer is affected when voids form beside or below the structure. Water filled bladders buried in the vicinity of the test structure were emptied to simulate erosion. These experiments demonstrated that loss of soil support besides or below the structure leads to decreases in vertical diameter (the test void below the pipe invert appeared to fill with soil from besides the structure, so the behaviour resembled that of the tests were support was removed directly from the sides).

Tan and Moore (2007) have used finite element analysis to study the effect of soil voids on the performance of rigid pipe culverts and sewers, Figure 305. That work demonstrated that the loss of soil support over a 90° angle at the springlines was sufficient to double or triple the bending moments, which could be expected to crack most rigid pipes (which are designed against excess moment using a factor of safety of about 2).

Moore (2008) has recently outlined the potential consequences of backfill erosion on liner design, providing design charts that indicate the effect of erosion on liner deformations, local bending, and buckling under external water pressure.

Figure 305. Soil void adjacent to culvert (Moore, 2008).

3.2.4.2_1g Inclusion of Vehicle Load Effects

No guidance is yet available for the manner in which vehicle loads should be incorporated into such strain calculations, though it should be possible to consider pipe deformation resulting from surface loading in these calculations for soil self weight. A calculation method incorporating the effect of vehicle load should be based on the deteriorated soil condition. Moore (2008) describes how average degraded soil modulus can be calculated for a fractured, deformed pipe, and perhaps flexible pipe theory with this deteriorated soil property could be employed in a calculation of the vehicle load effects.

3.2.4.2_2 Ungrouted Repair of Damaged Flexible Pipe

There is no known experimental study of the performance and behavior of these systems. There are no known finite element studies of these systems, except those discussed during the earlier sections dealing with buckling under external fluid loads. There is clearly a need to study ungrouted repairs to corrugated steel structures. Ideally that study will have both experimental and computational components.

Some designers consider the polymer liner placed directly in the soil and ignore the presence of the old structure. The expectation is that this neglects the old structure which still retains stiffness, so it should produce a conservative liner design. However, the work that has been performed on rigid pipe structures has clearly shown that it is not conservative to neglect the old pipe, Law and Moore (2007b). This is because there are strain concentrations where the liner spans across fractures in the old pipe, and local effects could also influence liners placed within deteriorated flexible structures.

To date, there has been no study of the effect of erosion voids on the design and performance of liners installed within flexible pipes. Some designers use the ASTM F1216-07b approach discussed below.

3.2.4.2_3 Grouted Repairs and Other Composite Systems

McAlpine (2009a) discusses the design of grouted liner systems under the influence of earth loads. He explains that the composite action of the liner-grout-host pipe system needs consideration, and includes checks of cracking in the grout and extreme fiber stress or strain in the polymer lining. He stresses the important distinction between behavior of bonded and non-bonded liners (extreme fiber stresses are substantially enhanced where bond develops). These recommendations and observations are important input for the work needed to repair culverts using grouted liners.

3.2.4.2_4 Design Approach in ASTM F1216-07b

Liner design for earth load as outlined in the current ASTM standard for CIPP liners works to prevent buckling as a result of earth loads. This approach results from standard developers who understood that they did not know which limit state to address, but knew that fluid load buckling was important and concluded that earth load buckling should also be addressed. However, the experimental and computational work of Law and Moore (2003; 2007b) outlined in earlier sections has demonstrated that thrusts do not develop in polymer liners placed within damaged rigid pipes. For these cases, therefore, buckling cannot occur (buckling requires thrust). Further work is needed to assess whether the thrusts that could develop in liners where they cross perforations in metal (at the waterline for example) or other culverts, can actually produce buckling instability.

A proposal is currently being developed for consideration by ASTM committee F17 to adopt the design methods of Law and Moore (2003; 2007b) and to remove considerations of liner buckling under earth loads for liners in rigid sewers. Instead, local bending strains caused by earth loads will be considered, Moore (2008).

3.2.4.2_5 WRc (2001) Design Approach for Earth and Live Loads

WRc (2001) suggests use of full overburden pressure on grouted liners, modeling them as a rigid pipe under the action of the overlying soil prism, therefore not requiring the composite repair to support loads considering the negative arching assessed during the design of rigid pipes buried in embankments, but not considering the benefits of positive arching or the load history (where the fractured host pipe would deform prior to repair, so that potential bending moments would be distributed away from the repaired system). The manual does indicate that "alternate" approaches are possible ("for example a very deep sewer constructed in a tunnel or heading"). A conventional vehicle pressure chart is employed, based on the work at the UK Building Research station in 1970. Distributions of pressure *P* with depth are considered for three different road classes.

Design uses crown bending moment calculated using

$M = C.P.d^2/4$,

for mean sewer width *d* and semi-empirical moment coefficient *C* (a function of coefficient of lateral earth pressure *K* and structural geometry, circular or egg-shaped). Tensile force in the liner is then calculated, though does not appear based on a rational evaluation of the composite action of the different components of the liner. The approach outlined by McAlpine (2009b) is superior.

WRc (2001) also provides an empirical value of the minimum bond strength required between the grout and the polymer liner, though the manual indicates that this is based on force in a 600mm sewer and indicates that it is conservative for larger structures.

3.2.4.2_6 Design Approach in ATV M127-2.

Falter (2001) provides details of structural design associated with German rehabilitation practice (ATV M127). This involves finite element analysis to assess local stress and strain resulting from earth and vehicle loads.

3.2.5. DESIGN FOR Floatation Forces

WRc (2001) provides guidance on the effect of the concentrated force that develops at the crown of a polymer liner during grouting (a reaction counteracting the buoyancy associated with the volume of displaced grout, minus the weight of the liner structure). The design manual presents graphs for calculation of the buoyancy force, and the 'deflection' that results in circular pipe liners manufactured from polyethylene with effective modulus of 350 MPa (about 50 ksi) and glass reinforced polymer. Pipes are selected to keep diameter change less than 6%. Material strain is also calculated in the GRP pipes to ensure it remains below 0.5%. These solutions need to be generalized for liners of other materials and geometries, and this should not be difficult.

3.2.6. GROUT-IN-PLACE LINERS

3.2.6.1 GIPL Design in Concrete Pipes

McAlpine (1997; 2007) discussed the design of grout-in-place liners (GIPL) in circular concrete pipes. The liner and the host pipe form a rigid structure, and the assumed failure state is flexural cracking of the grout (as opposed to the design assumption of buckling of flexible liners). The grout is the structural element while the plastic liner is of little, if any, consequence in the structural design (it provides corrosion protection).

Based on project designed choices and/or field limitations, GIP liners can be designed and installed as Type 1 (composites) or Type 2 (non-composites). In Type 1 design, (1) there must be adequate bonding between the grout and the host pipe, i.e., the bond is sufficient to ensure that strains are transferred from the host pipe to the grout, and (2) the grout and the host pipe are strain compatible, i.e., have similar elastic modulii.

3.2.6.1_1 State of Stress before Rehabilitation

The state of stress in the host pipe before the rehabilitation determines the location of the critical section.

Unreinforced concrete pipes (UCP) fail by flexural cracking at the springlines (outer surface) and at the crown and invert (inner surface) (Serpente, 1994). Watkins and Anderson (2000, p.416) gave formulae for calculating the bending moment and the thrust force at the crown and at the springline, from which the tensile stresses at the inner surface of the crown and outer surface of the springline can be calculated. These values can be compared with a nominal modulus of rapture in tension (implied from the compressive strength of the pipe material) thus indicating how close the current state of stress is to the flexural cracking stress.

Reinforced concrete pipe (RCP) does not fail by flexural cracking because the reinforced steel prevents pipe collapse until the ultimate load is reached. The current state of stress is calculated assuming that the steel takes the tensile force while the concrete (the remaining uncracked portion) takes the compressive force. The tensile stress in the steel can also be estimated from the crack width (i.e., multiplying the strain in the steel by the modulus of elasticity).

3.2.6.1_2 GIPL Composite Design (Type 1)

The host pipe and the GIPL are one structure. The composite material analysis requires the use of the transformed-section method of engineering mechanics. The design strategy is to increase the wall thickness at the critical section (at the crown of the host pipe). The added rehabilitation material (grout) does not experience any strain at the time of rehabilitation (current loads) but will reduce the flexural stress increase in the future (future loads).

The recommended design method is as follows (McAlpine, 1997):

- Compute vertical loads on the structure. Prism loads (WRc, 1990, p.III/40) can be used.
- Apply the calculated loads to the structure assuming uniform horizontal and vertical pressure distribution over full diameter of the pipe with K, horizontal/vertical pressure ratio, typically 0.4.
- Complete a structural analysis using transform-section method both before and after rehabilitation
- Design wall thickness and grout compressive strength to achieve required safety factor through iterations (analyzing different combinations of grout thickness and strength).

3.2.6.1_3 GIPL Non-Composite Design (Type 2)

The rigid host pipe and the GIPL are two separate structures (no bond between them). The GIPL doesn’t influence the failure of the host pipe (flexural cracking) and when the host pipe fails, the soil load is transferred to the liner as a concentrated load.

Assuming failure of the host pipe at the springlines and at the crown and invert, the host pipe deflects as four hinged segments until a new equilibrium with the soil is achieved. If the deflecting cracked host pipe reaches the GIP liner, it is assumed that there is vertical load (equal to the total vertical soil load) at both crown and invert of the GIPL but no horizontal load (the host pipe sides are moving away from the GIPL). The load condition is similar to the Three-Edge-Bearing (TEB) test for determining the strength of concrete pipes. The failure point of the grout will be at the inner surface of the grout ring of the GIPL at the crown and invert and at the outer surface of the grout at the springlines.

3.2.6.2 GIPL Design in Corrugated Metal Pipes

There are no published papers or guidelines about GIPL design in corrugated metal pipes and the following is the discussion from (George McAlpine, Danby of North America Inc, personal communication).

Buried CMP used as culverts are flexible structures with load characteristics and failure modes fundamentally different from rigid (e.g., RCP) culverts. The soil pressure distribution is likely to be uniform and radial, producing wall compression as the principal stress. Thus, CMP design is based on either wall crushing or buckling depending on the choice of wall characteristics (radius of gyration) and the culvert diameter or span (AISI, 1999). In many cases culverts in need of rehabilitation are significantly deformed either as a result of poor installation or subsequent unintended loads.

Type I GIPL (composite material) design can be used in CMP culverts but mechanical anchors MUST be used to insure strain transfer from the steel CMP wall to the grout. In the case of low deformation CMP, the principal strain/stress transferred to the grout will be compressive (its greatest strength). For highly deformed CMP, the compressive forces will produce bending moments that can result in flexural failure in the grout. Thus, both failure modes for the grout must be checked for cases of high deformation.

Type II GIPL design in CMP cannot share future compressive loads and, therefore, cannot prevent CMP wall crushing. Soil loads will only be transferred to the grout by deformation of the CMP after GIPL installation and predicting the nature of these loads is difficult. Use of non shrink grout will ensure intimate contact of the grout with the CMP wall and would resist circumferential shortening due to compression in the CMP wall and, thus, increase resistance to buckling. An intangible benefit of GIPL rehab in CMP, especially in large diameters that are normally constructed in place by bolting together structural plates, is the anchoring/stiffening of these bolted joints.

3.2.7. SPRAYED-ON LININGS

While there is a significant amount of data regarding curing times, temperature application, and basic mechanical properties (Table 1, Figure 1) of spray-on lining products, there is currently no dedicated nationally accepted standard in North America that provides guidelines to design the proper thickness of spray-on linings within a gravity pipeline (while some spray-on coating vendors used ASTM 1216-05 as a guideline, this standard was developed specifically for CIPP liners).

Table 37: Mechanical properties of a spray-on polyurethane coating product (Allouche and Steward, 2009)

Flexural Modulus	735,000 psi
Flexural Strength	14,000 psi
Long-term Design life (50 years)	70% of Flexural Modulus
Tensile Modulus	425,000 psi
Tensile Strength	7,450 psi
Elongation	2% at Failure
Density	87 pcf
Hardness-Shore D	90

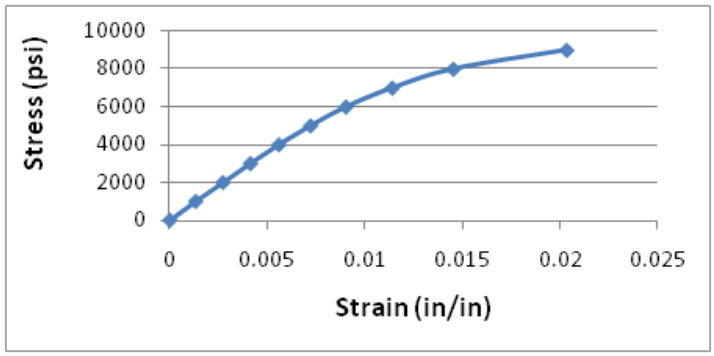


Figure 306: Nonlinear stress versus strain relationship of a spray-on polyurethane coating product (Allouche and Steward, 2009)

For high strength shotcrete common design procedure follows the compression ring approach

$$C = P_v \frac{S}{2}$$

Where C is the ring compression stress (psf), P_v is the total applied load (psf; typically H-80 loading adjusted to depth plus earth cover), and S is the pipe diameter (ft). The minimum required thickness of the shotcrete, *t*, required to resist the compression stress *C*, is calculated for the specific compressive strength of the grout (typically ranging between 2500 psi and 7,000 psi):

$$t = \frac{C}{(f_c' \times 144)} \times 12 \times N$$

Where *t* is the minimum thickness of the shotcrete (inches), C is the ring compression stress (psf), f'_c is the compressive strength of the grout (psi; typically at 7 days) and N is the factor of safety.

While the historical role of spray-on-liners in the buried pipe rehabilitation industry has been corrosion resistance and spanning of rust holes and small gaps in the wall of the host structure, there is little doubt that spray-applied liner contributes to the structural strength of host pipe. The question is the magnitude of this contribution and the relationship between the increase in increase liner thickness and the structural enhancement of the host structure. Specifically, in terms of hoop strength the large difference in the stiffness value between the typical steel reinforced concrete pipe or box culvert, and polymer lining will result in little structural enhancement as the liner will be carrying little, if any, stress. This might not necessarily be the case for CMP, where a still liner could contribute considerably to the stiffness of the host structure. A second open question is what happens to the liner when the host culvert experienced a sudden failure, as a well-adhered lining could tear if the host structure fractures. This literature review, as well as other recent studies, was yet to provide answers to these questions. The significant cost-saving potential associated with spray-on structural lining techniques for culvert rehabilitation making make an experimental examination of the utilization of spray-on structural liners for the rehabilitation of semi-flexible and flexible culvert structures a priority recommendation for Phase II of this study.

Sewerage Rehabilitation Manual (WRc, 2001) describes two design procedures (i.e., for Type 1 and Type 2 repairs) that are applicable for sprayed-on linings:

- WRc design of a type 1 liner is applicable to linings that adhere to the pipe being repaired creating a rigid composite pipe to carry all earth and live loads (e.g., man entry insitu coatings such as gunite).
- WRc design of a type 2 liner is applicable to linings that act as independent structures and do not adhere to the pipe being repaired (e.g., non man entry sprayed polyurethane and epoxy). The type 2 design concepts are applicable to both circular and non-circular linings.

Some standards exist applicable to the design of these liners in pressure (water) pipes. AWWA ***Standard for Cement-Mortar Lining of Water Pipelines*** (AWWA C602-06) describes design of cement-mortar lining of water pipelines, 4-in and larger, constructed of steel, ductile-iron, or cast-iron. Recommended thicknesses of cement-mortar linings, originally established in ANSI/AWWA C602-95, have been revised in this edition. AWWA standard, ***Spray-Applied In-Place Epoxy Lining of Water Pipelines*** (AWWA C620-07), describes design of an epoxy lining to the inside surface of previously installed water pipelines. The size of holes that cement mortar can span can be estimated based on structural calculations commonly used in the design of concrete elements. For a small hole a the governing failure mode is expected to be punching shear, while larger holes bending stresses are expect to govern.

Sprayed-on linings (cement mortar, epoxy, polyurethane coatings) are primarily used as spray-on protective coatings in case where the host pipe has structural integrity, and where infiltration and exfiltration are the issue. These linings are typically *nonstructural* (AWWA Class I) or *semi-structural* (AWWA Class II). Bontus et al. (2005) stated that in pressure pipes, with multiple layers application to the pipe wall named "High Build", Class II (semi-structural liners) can be achieved. High-build applications refer to formulation capable of 125-500 mil (3.2–12.5 mm) in a single coat. With maximum thickness of 25 mm or more for some solid polymer formulations.

Current research is looking into possibility that these liners could perform as AWWA Class III liners, have inherent ring stiffness and not rely entirely on adhesion to the host pipe.

Spray-on coatings have also been investigated for their capabilities and performance in a different yet similar application to the pipe relining: for rock support in mining applications (Archibald and DeGagne, 2001). Spray-on polymer linings have been observed to be capable of achieving significant area support resistance by virtue of their inherent stiffness, and to act as a potential shotcrete replacement in certain underground support situations. In large-scale block test, polyurethane coatings have been shown that they are capable of supporting the same load as #6 gage or even #4 gage welded-wire mesh as long as a continuous membrane of sufficient thickness is created (Espley et al., 1999).

Permacast® CCCP shell design methodology for underground pipe culverts provides the appropriate design calculations needed to determine the required liner thickness for lining storm culverts and sanitary sewer pipes under various levels and types of overburden loads (Henning, 2010).

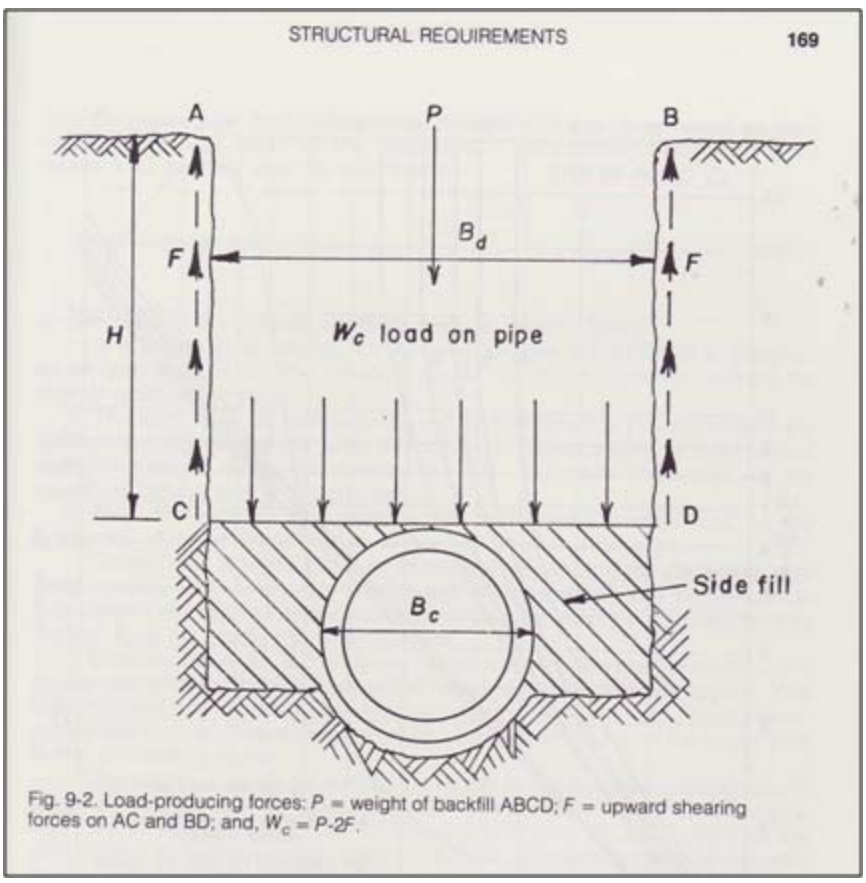


Figure 307: Loads in liner thickness design for centrifugally cast concrete pipe process (Henning, 2010).

It should be noted that nonstructural liners, which are designed assuming that they form a composite structure with the host pipe, rely on good mechanical bonding between the liner and the pipe (the mortar must fully penetrate defects in the pipe, joints, etc). The success of such applications using robotic spraying in non man-entry pipes depend heavily on the degree of surface preparation. Classes and II and

III sprayed liner must have sufficient inherent rigidity to stand-alone, with no reliance on bonding to the walls of then host pipe. These linings can be designed using ASTM F1216 as the CIP design equations in this standard do not consider bonding with the host pipe. Thus, the equations from this standard can be used to calculate a wall thickness for the structural lining required to stand alone against the groundwater pressure of a given depth.

In terms of surface preparation it is necessary to clean and profile the internal surface of the host structure prior to any spray-on application. Also, for most products all active infiltration must be stopped before the application and a relatively dry surface with adequate porosity is required to attain long-term adhesion. Steel culverts been coated should be prepared to a white or near-white metal according to NACE standards (NACE, 2006a, b). Corrosion spots in the steel surface should be reinforced using fiberglass patches for structural enhancement; either in the form of fiberglass fabric impregnated with resin or chopped glass fibers sprayed with resin. The National Association of Corrosion Engineers (NACE), the Society for Protective Coating (SSPC), and the International Concrete Repair Institute (ICRI) published guidelines dealing with proper selection of method of concrete surface preparation prior to coating and lining, or the installation of other protective system (ICRI, 1997). For concrete culverts, surface preparation commonly follows NACE No. 5/SSPC-SP 12 (NACE, 2002) and NACE No. 6/SSPC-SP 13 (NACE, 2003). All voids must be filled with high strength paste and jagged surfaces grinded. All surfaces that show steel reinforcement or spalling greater than ¾” deep should be patch. In areas where steel reinforcement is missing, steel replacement might be required (ASCE, 2010).

3.2.8. PIPE BURSTING DESIGN

3.2.8.1 Introduction

Pipe bursting involves the insertion of a conically shaped tool (bursting head) into the existing pipe to shutter the existing pipe and force its fragments into the surrounding soil by pneumatic or hydraulic action. A new pipe is pulled or pushed behind the bursting head. Design considerations associated with pipe bursting operations include the axial force needed to overcome the hoop strength of the host pipe, displace the soil around the bursting head and push the fragments into the in-situ soil, and pull the replacement pipe into the newly formed cavity. Design parameters include diameter, wall thickness and composition of the host pipe, degree of upsize (if any), length of pipe to be replaced, depth of installation (overburden stresses), the nature of the geological formation (sandy vs. clayey soil; normally consolidated vs. overly consolidated formation), moisture contact (i.e., above or below the GWT) and the presence of boulders/gravel beds. Problematic soil conditions include expansive soils and loose, dry sand. Other design considerations include ground movements (i.e., surface heave and possible damage to adjacent buried utilities and/or foundations), alignment correctness of the exiting culvert and the presence of concrete encasement, sleeves and/or point repairs. Additional considerations include local collapse of the host pipe and damage to the newly installed pipe from fragments of the host pipe (‘gouging’). In the case of CMPs and other flexible pipe materials a splitting head is utilized that cut the existing culvert along two or more longitudinal sections that are than displaced into the in-situ formation. The force needed to fragment the host pipe, displace the soil around the bursting head and push the fragments into the soil is substantial; thus, in many cases a reaction wall needs to be design to counteract this force.

The bursting head should be larger than the inside diameter of the existing pipe and slightly larger than the outside diameter of the new pipe to reduce friction and to provide space for maneuvering the new pipe. Pipe bursting is typically done in the range of 2” to 30” diameter pipes, although larger sizes can also be burst. Upsize of up to three pipe sizes (6” into 12”) have been completed successfully, but the larger the pipe upsizing, the more pulling force needed and the more ground movement will be

experienced. Usually a minimum depth of cover based on existing pipe diameter, the original trench conditions and the proposed upsizing is important to prevent surface and/or pavement heaving. The first step involving pipe bursting design is to evaluate the compatibility of a given culvert structure using pipe bursting technique and to address issues related to the host pipe and its soil environment. The geometry, material, and associated components (i.e., reinforcement, joint type, etc.) of the host pipe have a direct impact on the viability of the bursting methodology and the design of the expansion cone. Once it has been determined that the existing pipe is a candidate for bursting application, the structural condition, alignment and service connections (if any) must be assessed. Condition assessment generally results in existing piping systems falling into one of the two groups: (1) structures that are in a serviceable condition, but are, from a hydraulic perspective, undersized and require upsizing, and (2) structures that have some degree of deterioration that inhibits flow and/or dependability. Other considerations include launch and receiving pit locations, layouts and sizes, maximum pipe burst lengths, and equipment system capabilities. Prior to commencement of the pipe bursting process pre-design surveys should be undertaken, including land use (existing and future use), as-built drawings, site condition and surface/subsurface assessment, soil (geotechnical) information, utility location, environmental and social impacts and benefits, and cost estimate of the total project. Ariaratnam and Bennett (2005) outlined issues related to the preliminary design (method selection process, upsizing considerations, replacement pipe material, measures for protecting existing utilities, identification and assessment of risks), as well as in the design phase (upsizing factors, importance of volume displaced, soil displacement, pipe stress, and bursting force components).

To date there is no ASTM standard that provides a systematic procedure for calculating the stresses that are expected to develop within a pipe installed using the pipe bursting method, the required maximum pull force and the anticipated ground movements (i.e., heave and/or settlement). Nevertheless, several academics performed laboratory tests and developed analytical and numerical solutions for calculating the above mentioned quantities. Key aspects of relevant publications available in the public domain are summarized in the following sections.

3.2.8.2 Stress in the Product Pipe during Installation

Pipelines and culverts installed by pipe bursting are subjected to external pressure due to soil-pipe interaction and axial forces generated by the surface friction mobilized on the exterior of the pipe as it is pulled into position. Installation loads might be more severe than operational loads, and may govern the pulling length. Fernando and Moore (2001, 2002) proposed the use of the cavity expansion and contraction theory[3] proposed by Yu and Houlsby (1991; 1995) for predicting soil displacements around the host pipe during static pull and calculating external pressures on pipes being pulled into place. In pipe bursting operations any upsizing relative to the existing pipe causes permanent outward displacement of the soil around the bursting head. The zone of permanent displacement is called the plastic zone, and its outer dimensions depend on existing soil condition, initial cavity radius, and the expansion ratio (bursting head diameter divided by the outside diameter of the host pipe). Soil within the plastic zone reaches its yield stress, while soil outside the plastic zone remains in the elastic state. The greatest potential for damaging underground utilities is expected in the plastic zone due to relatively larger deformations. The displacement profile and the distance the plastic zone extends away from the exterior of the new pipe are both important to the safety of adjacent utilities. The mechanism of expansion and contraction of soil during the pipe bursting operation influences the magnitude of the

[3] Cavity expansion theory is concerned with the stress and displacement fields around internally and externally pressurized spherical or cylindrical cavities embedded in either linear or nonlinear media. The theory is detailed, for example, in Yu (2000) and Masri (2007).

radial stresses that develop between the soil, the original pipe, the bursting head and the pulled-in-place pipe. The axial friction that develops on the external boundary of the pipe during winching is influenced by the final radial stresses acting on the external boundary. The radial stress is also expected to influence local contact forces that develop between fragments of the original pipe fragments and the outer surface of the new pipe.

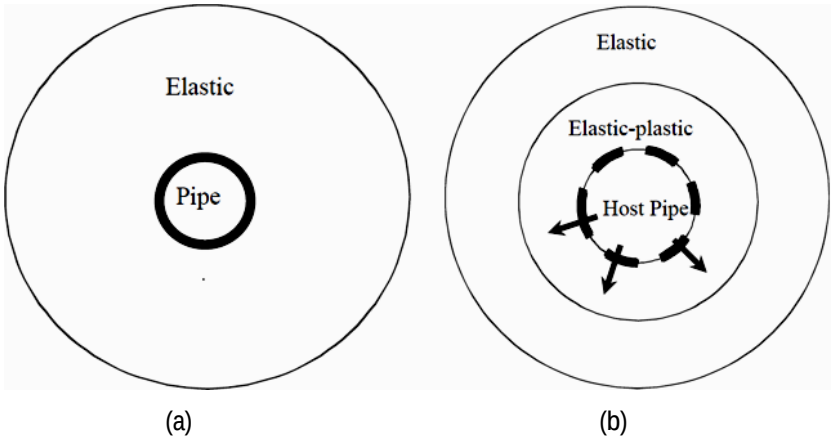


Figure 308: Ground conditions (a) before bursting and (b) during bursting (Fernando and Moore, 2001)

Gokhale et al. (1996) reported the development of an analytical model for calculating vertical and horizontal displacements in soil and loads on a new pipe installed by dynamic pipe bursting. Tests conducted in a full-scale testing facility in Germany showed that the loads on a new pipe installed by the dynamic bursting can be divided into two stages. In the first stage, the bursting head creates a temporarily stable cavity around the new pipe and the only loads on a new pipe are winching loads. In the second stage, the cavity breaks down, and the soil mass above the new pipe is displaced vertically downward forming a shear zone with dome-like section above the new pipe. (These phenomena are the result of dynamic traffic loads or alternation of ground water level.) With the presence of old pipe fragments in the ground, the ground pressure from the collapse of the cavity is transferred to the new pipe in the form of point loads. The equations showed a good correlation with the test results and provided design loads significantly lower than required by the German Standard ATV A161[4] (1990).

Cholewa (2009) evaluated the effectiveness of linear and nonlinear viscoelastic models (Moore and Hu, 1996; Zhang and Moore, 1997) for prediction of tensile axial strains during cyclic loading of HDPE pipe and showed that these models can serve as predictive design tools for estimating the cyclic strain history of HDPE pipe which was pulled into place. When compared against results from the experiments conducted on pipe samples, the linear and nonlinear viscoelastic models were found to provide reasonable estimates of the maximum strain levels during installation; however, maximum strains were underestimated by the linear viscoelastic model and overestimated by the nonlinear viscoelastic model. During periods of strain reversal, both models overestimated the amount of axial strain recovery. Using the nonlinear viscoelastic model to study axial pipe response under pure creep provided a conservative estimate of the maximum axial strain accumulated during a pull-in-place installation, provided the maximum pulling force is known. Creep and stress relaxation were shown to be prevalent conditions acting on HDPE pipes during and after a trenchless pull-in-place installation.

[4] German Standard ATV A161 (1990) applies for the static calculation of pipes with circular cross-section which are installed with static force according to the driven pipe procedure, with straight or bent alignment in non-cohesive or cohesive loose soils.

3.2.8.3 Required Bursting Force

The pulling force required for pipe bursting static pull can be attributed to three components: (1) the axial force required on the bursting head to break the existing pipe; (2) the axial force required on the bursting head to expand the soil cavity radially outwards and move the bursting head forward; and (3) the axial force mobilized because of friction between the replacement pipe and the broken host pipe.

Hahn and Ariaratnam (2003) offered a model for calculating the theoretical maximum pull force required to complete a static pipe bursting pull, which must be exerted by the pipe bursting equipment. The pull force is a function of friction, bursting and soil compression forces. Although the model was developed for the static pull, with some minor adjustments this model could be applied to the other methods, i.e., to the pneumatic or hydraulic expansion methods.

Typically the cavity expansion force is significantly larger than the breaking force and the friction force. In physical tests conducted in poorly graded sand, Lapos (2004) found for the specific conditions tested the cavity expansion force, breaking force and friction force were approximately 79%, 20%, and 1% of the total force, respectively. Pulling forces were also found to be dependant of the burial depth. Tests by Lapos were conducted in a 6.5 ft long, 6.5 ft wide, 5.2 ft deep test cell.

Allouche et al. (2010) performed a series of bursting tests of a VPC pipe buried in granular soil under varying overburden pressures. The work included laboratory tests utilizing a 6’ x 4’ x 12’ long soil-structure interaction chamber as well as a full-scale field installation aimed at quantifying the axial force needed to overcome the hoop strength of non-reinforced gravity sewer and drain pipes buried at different depths in granular soil conditions. A simulated pipe bursting operation was conducted in three phases in an attempt to quantify the required pulling loads needed for a small bursting operation between manholes. It was shown that the force required to burst the pipe is directly related the overburden soil pressure. For the materials and simulated depths used in the study, the portion of the total force used to overcome the hoop strength of the vitrified clay pipe (i.e. fragment the pipe) when buried in 15 feet of poorly graded sandy soil was 11.5%, while the remaining force was needed to displace the soil and overcome the friction between the replacement pipe and the soil medium. The force needed to displace the bedding material was found to be 330-450 lbs per linear foot for a 8-inch VCP pipe buried in saturated silty-clay under 5 foot of overburden, or 66 lb per foot of overburden/foot length for 8” VCP host pipe.

Nkemitag and Moore (2006) performed an axisymmetric finite element analysis to explore the axial progression of the burst head, the ground resistance to that advance, the magnitude of axial ground movements, and the effect of soil characteristics on the response of the burst head and the surrounding ground. Calculated response was compared to laboratory measurements of ground movement and pulling force to evaluate the effectiveness of the finite element modeling.

Nkemitag and Moore (2007) used axisymmetric finite element analysis to model the forward movement of the burst head along the pipe axis during static pipe bursting. Calculations were reported for the four pipe bursting configurations considered in an earlier laboratory study. Comparisons of calculated and measured values of pulling force reveal that the new finite element procedure provides effective measurements of the resistance of the burst head to forward progression and the total force required to pull the bursting head and new pipe through the old pipe system. The numerical procedure could be used in future studies to estimate pulling force requirements for specific projects, considering the burial depth, soil material, degree of pipe upsize, and specific geometry of the pipe bursting head.

Cholewa et al. (2009) quantified pulling forces in a well-graded sand and gravel, a material commonly used as sub-base beneath pavements. The tests consist of static burst of a 9 inch non-reinforced concrete pipe while pulling in a 9 inch outside diameter HDPEW pipe. The experiments were performed in a 26 ft long, 26 ft wide, and 10 ft deep test pit. The cavity expansion force was found to be between 22,500 and 30,000 lbs (approx. 77% of the total pull-in force), the force required to burst the pipe was found to

range between 4,500 and 11,000 lbs (approx. 22% of the pull force), while the friction force between the newly installed HDPE pipe and the formation was found to account for only 1% of the required fulling force (500 lb).

3.2.8.4 Identification and Assessment of Risk Associated with the Selected Method

The zone of disturbance or displacement created by pipe bursting operations is related to the installation geometry and ground conditions. In shallow ground operations upward movement will be noted during the expansion process, resulting in disturbance being localized and intensified above the bursting process. In the case of deeper grounds, the expansion zone is restricted to the vicinity of the pipe being replaced. If the soil conditions are relatively homogeneous, the expansion process will have radial distribution as the depth increases, and high radial pressure leads to volume compression of the surrounding soil. When a hard layer is below the expansion zone the resulting displacements will be predominantly upwards and will likely pass through a reduced volume of the soil to the surface. After reaching the maximum values during the expansion, the soil displacements start to recede, i.e. ground convergence towards the pipe axis start to take place. During the bursting operation an annular space is created behind the bursting head, the amount by which the bursting tool is oversized with the respect to the new pipe will control the size if this annular space, which should be minimized in order to reduce the ground displacement.

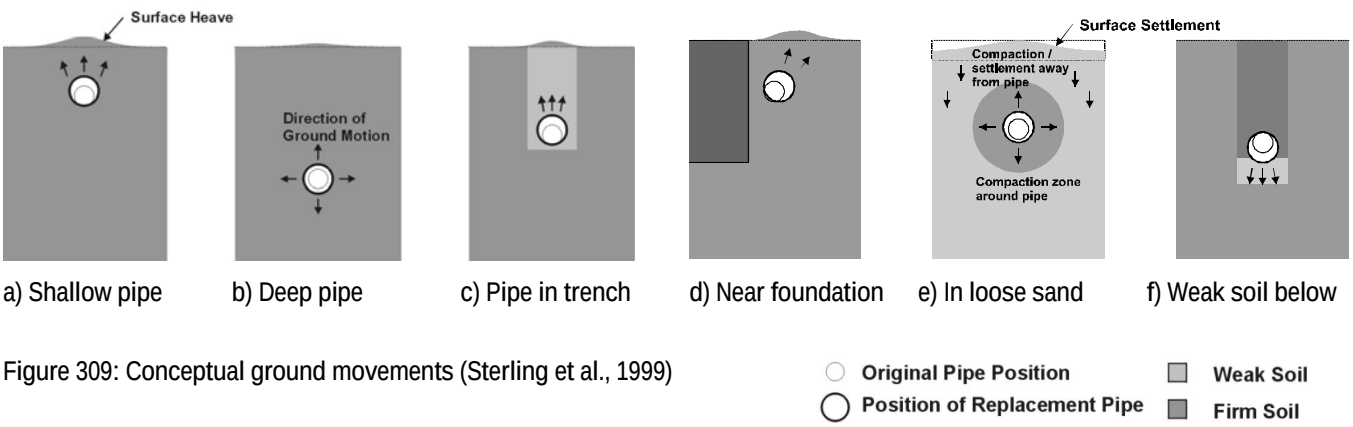


Figure 309: Conceptual ground movements (Sterling et al., 1999)

Atalah et al. (1998) reported field measurements of ground velocity and vertical movements during static pull bursting in four different soil conditions (clay, sand, silt, and a clay-gravel mixture) and concluded that ground displacements induced by pipe bursting depend on the degree of upsizing, type and degree of compaction of the soil, and the depth of cover above the pipe being replaced. Atalah (2003; 2004) studied ground movements associated with pipe bursting when the existing trench was dug in rock formations. In these conditions, pneumatic pipe bursting of large diameter pipes with high upsize percentage provides a higher level of ground movements and requires a longer safeguard distances from bursting head. The study could not correlate the level of ground vibrations with separation distance from the bursting head and was not able to recommend the safe distances for static pull.

Sterling et al. (1999) discussed ground movements associated with pipe bursting and the position of replacement pipe relative to the position of original pipe. Although the replacement pipe naturally follows closely the line and grade of the original pipe, the centerline of the replacement pipe only rarely coincides with the centerline of the original pipe because of variations in the resistance to the expansion movements in the soil caused by the bursting operations. Ground displacements depend on degree of upsizing, the type and compaction level of the in-situ soil around the pipe and the nature of confinement of the soil around the pipe. Figure 309 illustrates the types of ground movements that are probable under

various site and soil conditions. The new pipe's position is affected by entry alignment and stiffness of the new pipe, size of tail void and localized soil movements.

Fernando and Moore (2002) investigated the effect of soil parameters on ground movements in the vicinity of the static pipe bursting operations. A parametric study using axisymmetric cavity expansion theory showed that ground movements are controlled by soil strength and dilation angle rather than the elastic soil properties.

The following sections summarized observations reported during various studies which are related to risks associated with various practices utilized during the pipe bursting installation process or with specific subsurface conditions:

1. When a stiff new pipe is given insufficient room in the insertion pit to be aligned with the host pipe, the alignment of the host pipe might deviate downward from the original grade near the insertion pit.
2. If an existing sag in the original pipe has been caused by a soft zone beneath the pipe, the new pipe might be forced towards the soft zone by the bursting operation. Sags will be reduced if the surrounding soil is uniform.
3. If there is significant amount of sediment present at the invert of the host pipe the bursting head will move upwards relative to the existing pipe.
4. A rock base inhibits the breakage of the underside of the pipe and may cause the bursting head to deviate upwards from the centerline of the host pipe.
5. Existing utilizes located within a distanced equal to or less than two pipe diameters of the host pipe to be burst should be locally excavated to provide stress relief, an eliminate possible damage during the bursting process.
6. Ground vibrations generated during the bursting operations may be quite noticeable to a person standing on the surface, but are very unlikely to be damaging to a buried utility except at very close distanced to the bursting head.
7. The presence of broken pipe fragments causes the ground pressure to act as point loads on the newly installed pipe; this in turn causes higher stress concentrations. Design of new pipe should not only involve ground pressure but also effects of broken old pipe fragments.

3.2.9. HYDRAULIC PERFORMANCE

3.2.9.1 Hydraulic Design Issues

The hydraulic capacity of the rehabilitated culvert should be checked in the rehabilitation design phase to make sure that it will remain adequate after rehabilitation or be sufficiently increased if culvert replacement with upsizing is considered.

While a reduction in pipe diameter occurs as a consequence of relining, it is compensated to some degree by the reduction in the Manning flow coefficient (Table 38). If a culvert with a corrugated interior wall profile is lined with a smoother walled pipe (e.g., PVC, HDPE), the relined culvert flow capacity may even increase over that of the host pipe. However, the improved Manning coefficient affects the discharge capacity of the culvert if the culvert is long (i.e., when the energy loss is dominated by friction) and the culvert operates under outlet control.

Table 38: Typical Manning flow coefficients for water flowing through common culvert materials (from PPI, 2006b)

Piping Materials	Manning Flow Coefficient		Piping Materials	Manning Flow Coefficient
Corrugated steel pipe (CSP)	0.023		Cement-lined ductile iron	0.012
Concrete	0.016		Polyethylene (solid wall)	0.009
New cast iron, welded steel	0.014		PVC	0.009

For shorter culvert lining projects, enhancing the inlet geometry may have a measurable influence on the discharge capacity of the culvert, regardless of inlet or outlet control, particularly when the inside diameter decreases after relining compared to the diameter of the host pipe. For illustration, two different types of projecting liner end treatments with sliplining are shown in Figure 310, where the length of liner protrusion P can vary.

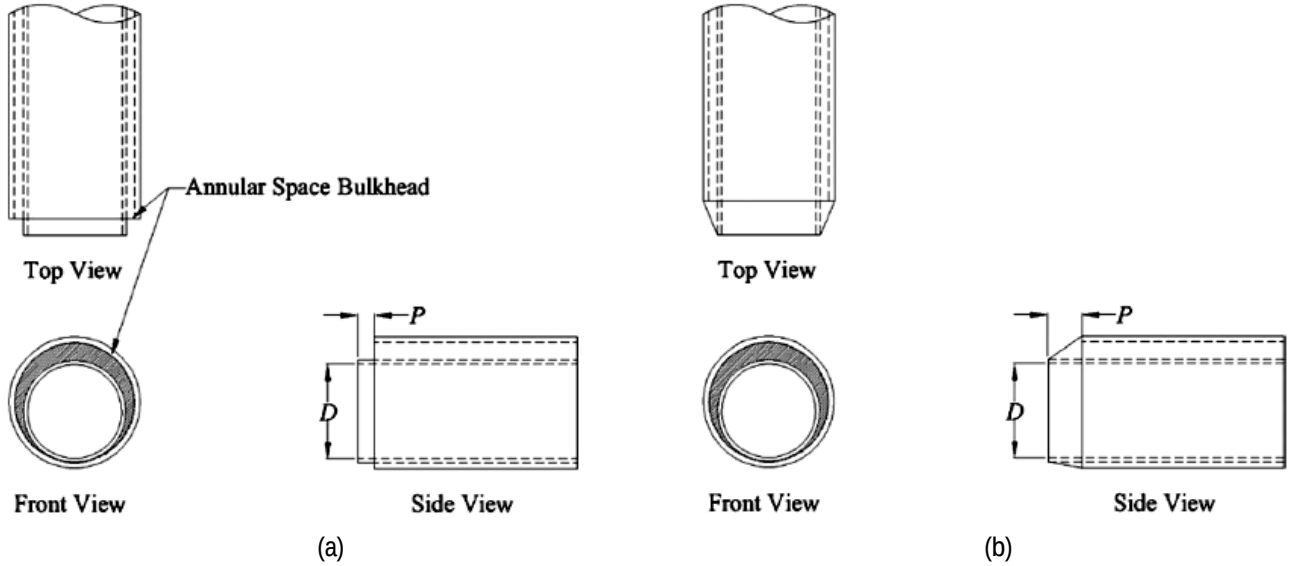


Figure 310. Two projecting liner end treatments: (a) non-tapered ("thin-wall") and (b) tapered (Tullis and Anderson, 2010)

The effect of inlet geometry on discharge capacity of culverts is especially important for culverts that operate under inlet control, where the head-discharge relationship is controlled by the inlet geometry and the culvert flow capacity is likely to be reduced after relining due to the reduction in inlet size. Tullis and Anderson (2010) evaluated experimentally (a) head-discharge relationships $Q/AD^{0.5}$ under inlet control and (b) entrance loss coefficients k_e under outlet control, for different projecting end treatments.

3.2.9.2 Hydraulic Analysis Calculations

With new cross-section area (reduced if relining or enlarged if replacing with upsizing) and changed flow conditions inside the pipe (different surface roughness inside the pipe), the hydraulic capacity can be calculated using culvert design charts in the FHWA's ***Hydraulic Design of Culverts*** (Norman et al, 2005), as discussed in Chapter 2.2 (*Hydraulic capacity of culverts*) of this report. In addition, various computer programs have been developed for simulation of the flow through a culvert to aid hydraulic analysis, which is complicated when using hand calculations. Although these programs incorporate the hydraulic theory outlined in FHWA's *Hydraulic Design of Culverts*, they implement the theory into the code slightly differently resulting in different headwater depth, flow control, and outlet velocities. Thiele (2007) and Hotchkiss et al (2008) evaluated seven computer programs[5] reviewing their features (e.g., ability to analyze roadway topping, multiple identical barrels, inlet and outlet controls, full flow option, hydraulic jumps, etc) and compared their results with calculations outlined in the FHWA's *Hydraulic Design of Culverts*.

[5] *HY-8 (FWA); Fish Xing 3.0 (U.S. Forest Service); Broken-back Culvert Analysis Program - BCAP 3.1 (Nebraska Dept of Roads); Hydraflow Express 1.07 (Intelisolve); Culvert Master 3.1 (Hacstad Methods); Culvert 2002-2 (TX DOT); and Hydrologic Engineering Center River Analysis System - HEC-RAS 3.1.3 (U.S. Army Corps of Engineers).*

3.2.9.3 Case Studies of Rehab Hydraulic Performance Evaluation

Najafi et al (2008) investigated hydraulic performance of CIP relined corrugated metal culverts near South Haven, MI. The overall approach was to compare the hydraulic characteristics of the culverts before and after relining (i.e. CMP vs. CIPP). The CIP's wall-roughness and cross-sectional geometry were measured first. Hydraulic performance of the culverts was analyzed for the projected 100-yr peak flow conditions in the creek (i.e. design flow 850-cfs). It was established that the hydraulics of culverts are governed by outlet control. The CIPP was shown to reduce the cross sectional area and the height of the barrel, but also to produce a significantly lower roughness than the recommended value for the original CMP culvert. The CIPP's reduced roughness lowered the headwater depths.

McGrath et al (2009) reported about CIPP rehabilitation of twin 72-in corrugated metal pipes (CMP) in Clay County, FL. Hydraulic modeling showed that the low friction of the CIPP pipes would have a greater hydraulic conveyance capacity than the existing corrugated pipes. Published values for Manning's roughness coefficient show 0.023 for CMP and 0.011 for CIPP. This large difference is only slightly offset by the reduction in diameter of the lined pipes, from an original 72-in to approximately 70.5-in. The estimated increase in flow capacity (approximately 40% increase in peak flow conveyance for the 25 year/24 hour design storm) would be taken as a hydraulic benefit in most water/wastewater applications but could create problems downstream in a stormwater application. To achieve a hydraulically equivalent system, stainless steel flow restriction plates covering a portion of the top of the CIPP pipe openings were required at the CIPP outfall (Figure 311).

Figure 311. Flow restriction plates at outfalls after CIP lining of CMP pipes (McGrath et al, 2009)

3.3. GUIDELINES FOR THE SELECTION OF REHABILITATION METHOD (TTC)

Based on the results of a national survey (56 responding agencies), the **NCHRP Synthesis 303** *Assessment and Rehabilitation of Existing Culverts* (Wyant, 2002) indicated that a very small number of agencies had guidance to select culvert repair methods (9% of responding agencies) or rehabilitation methods (7% of responding agencies) as of 2002. Of the responding agencies, 27% considered the following six factors in their decision to either replace or rehabilitate culverts: hydraulic capacity, structural capacity, volume of traffic, height of fill, remaining culvert service life and risk assessment (i.e. probability of failure and severity of consequences). Most of these agencies factored in the service life of the culvert into their decision making process (24% responding agencies).

The NCHRP Synthesis 371 (Markow 2007) presented another national survey (39 responding agencies) that indicated by 2007, a growing number of agencies had guidance for maintenance and rehabilitation of culverts (the survey in 2002 did not cover maintenance). Although national guidelines exist, such as FHWA's *Culvert Repair Practices Manual* (Ballinger and Drake 1995), AASTHO *Culvert Inspection, Material Selection and Rehabilitation Guideline* (AASHTO 1999), and AASTHO *Maintenance Manual* (AASHTO 2007), guidelines issued by individual agencies were identified as the primary technical source for culvert inspection, maintenance and repair.

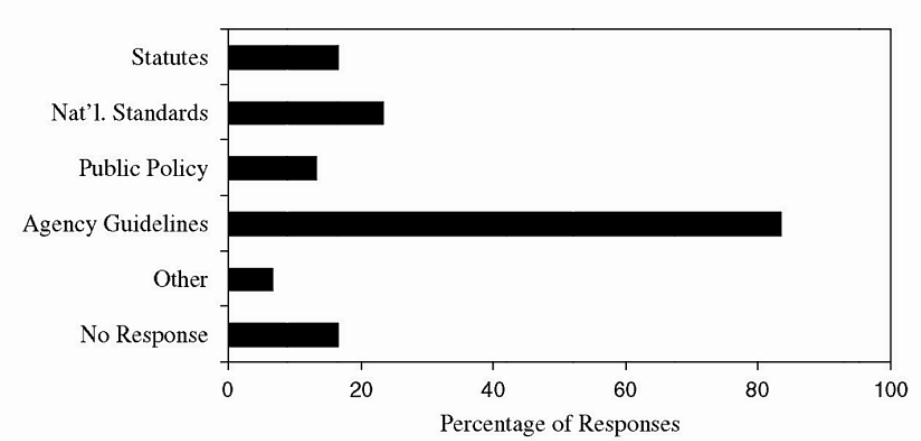


Figure 312. Technical management guidance for maintenance and rehabilitation of culverts. (Markow 2007)

The guidance system used by the Minnesota DOT investigated seven different culvert rehabilitation options in concrete and corrugated metal culverts (Table 39) in an effort to find an inexpensive and less disruptive alternative for the removal and replacement of deteriorated culverts (Johnson and Zollars, 1992).

Table 39. Scope of the MnDOT's study: Rehabilitation options and criteria for their comparison

Method	Product installed	Criteria
1 Sliplining	Smooth polyethylene with mechanical joints (PEM)	▪ Cost ▪ Skills and resource requirements ▪ Time requirements ▪ Culvert preparation ▪ Traffic disruption ▪ Work area requirement ▪ Placement problems ▪ Grouting procedures
2 Sliplining	Smooth polyethylene with fused joints (PEF)	
3 Sliplining	Corrugated polyethylene (PEC)	
4 Sliplining	Spiral ribbed polyvinyl chloride (PVC)	
5 Sliplining	Fiberglass (FG)	
6 Sliplining	Spiral ribbed coated steel arch (ST)	
7 CIP liners		

The two-volume FHWA's *Culvert Repair Practices Manual* (Ballinger and Drake, 1995) outlined different culvert repair strategies, showing objectives and work options of each (Table 40). In addition to traditional open cut, the manual described several trenchless excavation methods (TEC) for the installation of new culverts. Three primary methods of economic analyses to help in selecting or prioritizing culvert projects were outlined: first-cost analysis, life-cycle cost analysis, and benefit-cost analysis.

The Caltrans' *Culvert Restoration Techniques* (Aryani and Al-Kazily, 1993) compiled similar information on rehabilitation and installation methods and techniques as the FHWA's *Culvert Repair Practices Manual* (referencing its 1992 draft). The manual also covered fold-and-form liners as a rehabilitation option. Two technologies that are capable of the trenchless installation of new culverts were pipe jacking (for pipes 48" in diameter or larger) and microtunneling (for pipes up to 36" in diameter).

Caltrans *Supplement to FHWA Culvert Repair Practices Manual* (2003) provides updated information on rehabilitation construction methods. Two of the newly covered construction methods are sprayed epoxy or polyurethane coatings, and man-entry relining with pipe segments (i.e. fiberglass reinforced cement (FRC) liners and fiberglass reinforced plastic (GRP) liners). Also, specific procedural guidance is provided on culvert performance and condition assessment.

Table 40. Types of culvert repair strategies (Ballinger and Drake, 1995).

Strategy	Objective	Work options
Routine maintenance	To keep a culvert in a uniform and safe condition by repairing specific defects as they occur.	▪ Debris & sediment removal ▪ Thawing frozen culverts
Preventive maintenance	More extensive strategy than routine maintenance intended to arrest light deterioration and prevent progressive deterioration.	▪ Joint sealing ▪ Concrete patching ▪ Mortar repair ▪ Invert paving ▪ Scour prevention ▪ Ditch cleaning, repair
Rehabilitation	Takes maximum advantage of the remaining unusable structure in a culvert to build a reconditioned culvert.	▪ Repair of basically sound endwalls and wingwalls ▪ Invert paving ▪ Repair of scour ▪ Slope stabilization ▪ Streambed paving ▪ Add apron, cutoff wall ▪ Improve inlet config. ▪ Install debris collector
Upgrade to equal replacement	Upgrade to provide service that is equal to that provided by a new structure.	▪ Addition, repair or replac of appurtenant structures ▪ Lining of the barrel. ▪ Provision of safety grates or safety barriers ▪ Lengthening of the culvert
Replacement	Provide a completely new culvert with a new service life.	Can be accompanied by: ▪ Realignment ▪ Hydraulic structural and safety improvements ▪ Change in culvert shape or material

The FHWA's *Culvert Pipe Liner Guide and Specifications* (Thornton et al., 2005) provides more in-depth information about many culvert rehabilitation technologies including method descriptions, advantages/limitations, cost information and general installation guidelines. Applicable standards and specifications are also listed, as well as contractors and manufacturers of systems used in the US market. Methods used for the rehabilitation of corrugated metal culverts include: sliplining (segmental and continuous), close fit lining (swage lining, rolldown lining, deform/reform lining, fold-and-form lining, spirally wound lining, and cured-in-place pipe (CIPP) lining) and spray-on lining (cement mortar lining and epoxy lining).

The Virginia DOT (VADOT, 2008) has developed its own guidelines for the selection of culvert rehabilitation methods (Table 41).

Table 41. VA DOT's flexible liner type selection guideline (VADOT, 2008)

Pipe Deficiency or Site Limitation	Concrete	Corrugated Metal	Plastic
Minor Cracks	A, B, C, D, E, F	NA	A, B, C, D, E, F
Major Cracks and/or Spalls	A, B, D, E	NA	A, B, C, D, E
Joints Separated >1 inch	A, B, C, D, E	A, B, C, D, E	A, B, C, D, E
Coating Removed, No Corrosion	NA	A, B, C, D, E, F	NA
Coating Removed, Minor Corrosion	NA	A, B, C, D, E, F	NA
Coating Removed, Major Corrosion	NA	A, B, C, D, E	NA
Minor Deformation, <5% of inside diameter	NA	A, B, C, D, E, F	A, B, C, D, E
Intermediate Deformation, 5% to 7% of inside diameter	NA	A, B, D, E, F	A, B, D, E
Major Deformation, >7% of inside diameter	NA	A, B, D, E	A, B, D, E
Height of cover	*	*	*
Access (Limited space to end of pipe, accessible by manhole or drop inlet)	A, B	A, B	A, B
Bends in pipe	A, B	A, B	A, B

* Note: An economic evaluation should be performed to determine the feasibility of excavating and replacing rather than lining the existing pipe	LEGEND: A;Cured In Place Pipe (Insituform, CIPP COR72AT470, Am-Liner, National Liner) B;Fold and form flexible liner (U - Liner) C;Thin walled HDPE slip liner (Spirolite, Snap-Tite, Danby) D;Thick walled HDPE slip liner (N-12, CONTECH A-2, CONTECH A-2000, Ultraliner) E;PVC slip liner (Easy Liner, Lamson Slipliner,) F;Spray - on polymer (Polyspray Full Structural, Poly-Triplex Liner System) NA;Not applicable

Syachrani et al. (2008) conducted a survey of state DOTs in the US to find out current practices for culvert maintenance and rehabilitation. Out of 20 responses received, nine agencies reported not to perform maintenance and rehabilitation of roadway drainage structures or culverts (AK, AR, HI, KY, LA, MS, NY, SD) whereas eleven agencies did (CT, FL, MN, NH, NJ, NM, OH, SC, TN, UT, VT) (Figure 313). The survey also identified the rehabilitation techniques used by the DOTs and the level of popularity (Figure 314). Each of the 20 responding agencies reported using sliplining, while many reported also using cured-in-place and close-fit relining (symmetrical reduction systems, fold and form, snap-tite, etc.).

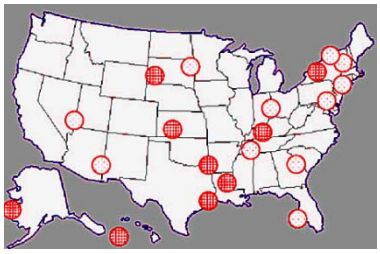


Figure 313. Survey of DOTs about culvert maintenance and rehabilitation practice (Syachrani et al., 2008)

Technologies	Degree of Popularity (Frequency of responses)				Pop. Index	Rank
	VP	P	U	VU		
Slip-Lining	9	1	1	-	93.2	1
Cured in Place Lining	1	4	1	-	75.0	2
Invert Repair	1	-	1	-	75.0	2
Close-Fit Lining	-	1	3	-	56.3	4
Spiral Wound Lining	-	-	-	3	25.0	5
Joint Repair	-	-	-	1	25.0	5

VP: Very Popular, P: Popular, U: Unpopular, VU: Very unpopular

Figure 314. Popularity of culvert rehabilitation methods (Syachrani et al., 2008)

The Code of Federal Regulations (CFR) 635.411 (FHWA, 2005) covers the material and product selection for culvert pipes. The FHWA implemented Section 5514 of the *2005 Safe, Accountable, Flexible, Efficient Transportation Equity Act*, titled "Competition for Specification of Alternative Types of Culvert Pipes" (US House of Representatives, 2005) by revising CFR 635.411 (the FHWA issued several policy memoranda between 2005 and 2007 to clarify the interpretation of material or product selection policy regarding culvert pipes). The current policy requires State DOTs to consider all available pipe products that are judged to be of satisfactory quality and equally acceptable on the basis of engineering and economic analyses. The FHWA has developed an informational web page titled “Construction Program Guide, Culvert Selection” (FHWA, 2008) for states that are still considering revisions in their culvert selection policies to provide for a wider application of culvert materials. The web page provides links to the Section 5514 statutory and regulatory requirements, FHWA informational memoranda, and links to AASHTO documents related to the design, materials and construction criteria for various culvert installations.

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