

# Committee Report



ASCE

*Pipeline Division*

**Pipeline Infrastructure (PINS) Task Committee**

American Society of Civil Engineers

## **Emerging Concepts for the Design of Pipeline Renewal Systems**

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# Emerging Concepts for the Design of Pipeline Renewal Systems

## FOREWORD

The Emerging Concepts for the Design of Pipeline Renewal Systems developed out of the efforts of the ASCE Task Committee authorized by the ASCE Pipeline Division's Pipeline Infrastructure Committee (PINS). It includes previously published ASCE papers presenting this topic.

During 1999 the PINS Committee formed the Task Committee for evaluating the current calculation methodologies for determining renewal liner design. It was determined by PINS Committee during the 1990's and culminating in 1997 that the calculation systems were conservative as employed. During January 1998, also an International Trenchless Technology Research Colloquium at Louisiana Tech University determined that a new calculation methodology should be developed.

The commonly used "Fully Deteriorated" calculation system was based on the model used for trench installation of Fiberglass Pipe. The evaluation of this model was of prime concern to the Task Committee. It was quickly determined that the so-called "Fully Deteriorated" concept should be revised and basic fundamentals be researched, reviewed and incorporated into a proposed report. This task was completed and a synopsis is provided in this Committee Report.

Several international and domestic experts directly or indirectly provided input that is shown in this report. An independent "Blue Ribbon Committee" selected by the Pipeline Division has reviewed the contents of this report to ensure an acceptable consensus of industry professionals working in the pipeline rehabilitation (renewal) area, i.e., research, design, and construction, is presented.

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## Disclaimer

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# PART I

## GENERAL

### 1.0 Introduction

A conventionally buried pipe is structurally designed to “carry” or “transfer” all service loads, which includes the weight of soil backfill placed over it, hydrostatic pressures, vacuum, internal working pressures, and loads applied at the ground surface. More than 60 years ago, Spangler demonstrated that flexible pipe which had insufficient stiffness to support the vertical soil load on its own, nevertheless performed well when buried because it deformed and developed passive lateral support. With the advent of renewal techniques using plastic pipes, this simple load/response concept prevailed and, for lack of any other available design methodology, engineers in many countries simply assumed that in the limiting case of a badly deteriorated host pipe, the renewal system would have to “take over” the vertical soil load and respond in the same way as a conventionally buried flexible pipe. It has led in turn to the building of layer upon layer of over conservatism in the calculation of renewal wall thicknesses for the so-called “Fully Deteriorated” design case. As will be described in future sections, this assumption is fundamentally questionable. Therefore, revised design recommendations for pipeline renewal systems are provided in this Committee Report.

### 2.0 Renewal Earth Loads

Generally, loads on a buried pipe are difficult to estimate with a high degree of accuracy and are often grossly overestimated in design. Some Engineers would question the term “fully deteriorated” as a poor choice of words. A definition of “fully deteriorated” would imply that the structure is no longer capable of carrying its load and has failed, i.e., collapsed. Obviously, no liner could be installed in a pipe that is truly “fully deteriorated,” and it is collapsed.

Failure modes for rigid pipe are well known. Corrosion or loss of cementitious material in wall thickness and/or loss of steel reinforcement cross section result in cracking of the wall followed by minor deflection until in some cases the steel reinforcement ruptures. Four hinged cracking of other rigid pipes can also occur. In either case the pipe-soil structure acts as a flexible pipe with deflection increasing until the pipe and soil reach a new equilibrium condition. The amount of deflection primarily depends on the soil conditions, the uniformity of support and the soil stiffness (modulus of soil reaction). If the deterioration of the pipe structure is uniform around the inside surface of the pipe, which is unlikely, the pipe will usually crack at the crown and invert on the inside surface and at the springlines on the outer surface. If the deterioration is primarily in the top of the cement based rigid pipe, as is common in sewers, the invert is not likely to crack unless it was cracked to begin with. In the case of other non-cementitious pipes the cracking will increase slightly until a new equilibrium is reached.

The installation of a renewal liner ceases the bio-sulphuric corrosion and/or soil movement through the pipe wall into the pipe. Therefore, further interior deterioration of the rigid pipe stops and the so-called “fully deteriorated” condition does not occur. The surrounding soil never comes into direct contact with the liner in any case so the liner soil interaction does not occur as has been assumed.

The inherent stability of old pipeline structures, even where the pipe element is badly deteriorated, was recognized in the original renovation design concept published by the Water Research Center (WRc) in 1983, and still applies in the UK today. It was recognized that rigid pipe which has lost its ring stiffness by longitudinal cracking and/or corrosion would deform like a flexible pipe to establish a new equilibrium with the surrounding soil. The degree and nature of the deformation is dependent on the quality of the original installation, pipe material properties, surcharge loads, in situ conditions and backfill stability. Infiltration of groundwater through the cracks or openings could eventually destabilize this equilibrium by washing soil into the pipe. It was assumed that a renewal liner would stabilize the host pipe condition. Considering no additional surface loads were applied, there is no reason to believe any soil load to act on new system. Therefore, it can be concluded that a liner pipe can be safely designed to resist external hydrostatic pressure only (See International Synopsis).

Further, recent experimental research (Gumbel et al., 2003) has shown that uniform soil pressure on the liner pipe, as required in the current ASTM F1216 and ASCE Manual of Practice No. 62 on design practice, is improbable and use of the current design equations for the “fully deteriorated” condition is inappropriate. Any soil load transferred to the liner will produce bending and little, if any, thrust loads necessary for buckling (See also Part II of this report).

### **3.0 Renewal Hydrostatic Buckling**

It is a safe assumption that any pipe requiring renovation will not be leak-tight, and so all renewal systems are potentially exposed to external hydrostatic pressure due to groundwater plus any internal vacuum pressure that could develop during installation or in service. Current design practice (ASTM F1216) and its design equations have limited predictive power because of lack of theoretical consistency as will be explained later in this report. A fully consistent design theory that is widely accepted is presented in Part III of this report.

Since groundwater pressures may be subject to unpredictable seasonal variations, especially on deeply buried pipelines, both peak short-term conditions and long-term conditions with their durations must generally be considered.

### **4.0 Sliplining Annulus Grouting**

In-depth studies and field experience on sliplining annulus grouting have proven that when flotation is minimized and grout pressure is maintained at 20 to 25 percent of the liner pipe stiffness, the pipe is safely below its critical buckling pressure. This grout

pressure criterion assumes the liner deflects approximately three (3) percent or less during the grout placement. While the actual pressure on the liner pipe is less than the gage pressure, the precise value cannot be determined. Therefore, this level of conservativeness is suggested and commonly used in the industry.

In most personnel-entry pipelines, the linings can be “blocked off” circumferentially at relatively short distances and/or grouted in lifts. In non-personnel-entry pipelines, the grout mix normally requires low density control and maintaining full flow condition within the pipeline during the grouting operation. This topic will be covered in future reports.

## **5.0 Summary**

As previously discussed, the rigid host pipe-soil structure is the dominant soil load bearing resisting structure and must be assumed to be in stable equilibrium or will usually attain such equilibrium with only small deflections of the host pipe. When considering this rationale, the concept of the host pipe being “Fully Deteriorated” may become inappropriate. Thus, this report recommends that liner pipe design should be based on hydrostatic buckling, using the proposed new design equations of Part III of this report.

Where there are good reasons to presume that some vertical deformation of the existing pipe/soil structure could occur after renewal (although as previously discussed, this is most improbable), it is important to realize that the ring stiffness of even a thick renewal system is unlikely to have any impact in resisting that deformation. The renewal system stiffness is likely to make only a small contribution compared to the soil stiffness for most soils involved in pipeline renewal projects. The reduction of unnecessary renewal system thickness is not only an enhanced design procedure but it can also be a cost reduction feature. Vertical deflection (if any) should be included in the hydrostatic buckling design of Part III as an imperfection.

The ASCE Pipeline Division’s PINS Task Committee on Pipe Renewal Design recommends consideration of the design methods contained in the following parts and described in this report in all future engineering designs for pipe renewal

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Task Committee Chairman

## PART II

### REHABILITATION OF FULLY DETERIORATED RIGID PIPES BY FLEXIBLE AND RIGID LINERS<sup>1</sup>

#### 6.0 Introduction

Since 1997 various technical groups, including an ASCE PINS Task Group, have been discussing changing the design guidelines for the rehabilitation of “fully deteriorated” pipes by flexible liners. A paper by Schrock and Gumbel at No-Dig '97 (Schrock and Gumbel, 1997) presented a strong case for revising the current design practice, especially the way in which earth loading is determined. This subject has been the topic of discussion at four symposia sponsored by the Pipe Rehabilitation Council (PRC) over the past four years.

Current practice in the USA for designing pipe rehabilitation liners for the “fully deteriorated” condition (see ASTM F 1216, for example) is to use the full prism earth column as the vertical load on the pipe/liner. Further, the assumption is made that the host pipe is replaced by soil and the design follows a modified buckling equation developed for the direct burial of fiberglass pressure pipe (Fiberglass Pipe Design, AWWA Manual M45). The modification introduces an ovality factor that is not included in Manual M45. The definition of “fully deteriorated” is that the host pipe is, or will become, incapable of carrying its structural load (earth, water, and live loads). The above design approach assumes that ignoring the presence of the host pipe and designing the liner as a directly buried flexible pipe capable of carrying the structural load specified is usually a “worst case” design. Often the interpretation of this definition is that the designer must assume the host pipe has zero structural capacity. This part discusses some logical inconsistencies in this design method and raises questions regarding the “worst case” assumption.

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Schrock and Gumbel (1997) argue convincingly that the soil surrounding the deteriorated host pipe being renewed has stabilized and consolidated over the extended time since its burial many years ago. Thus, it is reasonable to assume that the soil load on the pipe/liner is much closer to a tunnel load than either a trench or embankment load. Therefore, given the soil type (cohesion and friction) the designer can calculate the arching factors that are used to multiply the vertical soil column (prism load). For tunnels these factors are less than 1.0, their actual value being strongly dependent on the soil properties. Other than to point out that this argument may not be valid in certain soils, e.g., cohesionless soil; this part will not discuss this topic further. Because this part is largely concerned with the appropriateness of the engineering mechanics model of the physical phenomenon of the actual deteriorated pipe-liner-soil system, a brief review of the predictable ways that various types of pipes fail is in order. Serpente (1994) discusses the stages of deterioration of clay, brick and concrete pipes, the principal materials used for sewers now in need of rehabilitation. Due to space limitations, only the concrete pipe deterioration process will be discussed here.

Serpente (1994) gives the following descriptions of the three stages of deterioration of concrete pipe:

Stage 1 - "Pipe cracking is caused by bad laying practice, or subsequent overloading, or disturbance. The sewer remains supported and held in position by the surrounding soil. Visible defects: cracks at soffit, invert and springing. Infiltration may also be visible."

Stage 2 - "Infiltration of groundwater or infiltration/exfiltration caused by surcharging of the sewer washes in soil particles. Side support is lost allowing further deformation so that cracks develop into fractures. Side support may also be insufficient to prevent deformation if the original backfill was either poorly compacted or of an unsuitable material. Visible defects: fractures, slight deformation. Infiltration may or may not be visible."

Stage 3 - "Loss of side support allows side of pipe to move further outwards and the soffit to drop. Once the deformation exceeds 10%, the pipe becomes increasingly likely to collapse. Visible defects: fractures and deformation, possibly broken."

Reinforced concrete pipe (RCP) strength is rated (ASTM C 76) in terms of its D-load to produce 0.01 in crack and its D-load to produce the ultimate load. The ultimate load indicates failure of the steel reinforcement while the 0.01 in crack indicates that the tensile stress at the surface of the concrete due to bending moments has exceeded the tensile strength of the concrete. The specified ultimate D-load is, for most pipe classes, 50% greater than the 0.01 in crack D-load. Clearly the surface cracking is not considered a failed pipe condition. In fact, the classical indirect design method for RCP (pp43-48, Moser 1990) uses the 0.01 in D-load strength as the acceptable service load relying on the 50% strength margin to ultimate load as the required safety factor. This same design method allows vertical loads to exceed the cracking D-load strength by as much as a factor of 3, depending on the type of bedding employed. The bedding process involves the placement of soil or other bedding material under and around (up to springline) the pipe. This soil support lowers the magnitude of the bending moments relative to those of

the D-load test (three-edge-bearing test, ASTM C 497). Clearly, disturbance of the bedding material (or improper installation) can lead to pipe failure. It should be noted that the deformation (deflection) of the RCP at cracking is much less than 1% (Watkins 2000, p. 69). Similar observations hold for non-reinforced concrete pipe except, of course, the ultimate strength is equal to the cracking strength.

## **7.0 Fully Deteriorated Pipe**

First the logically flawed term “fully deteriorated,” as it applies to the condition of the host pipe being rehabilitated, must be dealt with. A truly fully deteriorated pipe has, by definition, collapsed; it was not capable of carrying its structural load (the final phase of Serpente’s Stage 3). Liners under discussion here cannot rehabilitate such pipes. Thus the only logical assumption left would be that the fully deteriorated condition would be attained after the rehabilitation liner is installed. Note that the structural safety factor of the host pipe-soil structure is, at the time of rehabilitation, equal to or greater than 1.0. For the pipe to become “fully deteriorated” it must either lose structural capacity or its structural load must increase.

Further structural capacity deterioration of the host pipe after rehabilitation can occur only if the liner fails and allows chemical attack of the host pipe wall or pre lining cleaning fails to remove corrosive chemicals from the pipe walls. This assumes that the physical deterioration of the pipe occurs from chemical attack inside the pipe as opposed to attack from outside at the pipe-soil interface. With normal precautions in job specification and liner design, further deterioration of structural capacity is unlikely.

Increases in the structural load on the lined pipe can occur by increased vertical soil loads, such as adding a heavy structure at the surface above the pipe. As the load calculation for ASTM F 1216 is the total load just prior to lining, the design method does not account for future increases in loading. According to Serpente (1994), the most likely scenario producing structural failure of the rigid host pipe (i.e., deflection) after lining would be loss of soil side support due to ground water movement (e.g., soil being carried into pipe by infiltrating water, etc.). This increases the bending moments that may lead to tensile failure of the rigid pipe without any increase in vertical load. Of course, if the vertical load was determined using tunnel conditions, soil erosion may well increase the vertical load by destroying the soil arch and invalidating the tunnel assumption.

## **8.0 Liner-Pipe-Soil Structure**

Due to the large difference in elastic modulus between most rehabilitation liners and their host rigid pipe, very little of the pre-cracking load will be carried by the liner (even if bonded to the pipe wall). One could assume that when the rigid pipe fails and deflects, it reaches a new equilibrium with the soil. However, the pipe-soil structure is still carrying the principal (perhaps, the full) load with the liner carrying little, if any, of the earth load. Thus, the liner will carry the full earth load only when the host pipe fully collapses (separates at the hinges) and is no longer acting as a unit but as four separate

pipe quadrants. This is the geometrical configuration of the liner pipe when it finally carries the full earth load calculated by F 1216: over 20% deflection with rigid pipe fractured edges pressing into the liner at the crown and invert with full earth load pressure! Of necessity, we must assume that the soil has allowed this movement of the pipe walls. That is, voids or loose soil that can be compacted to allow over 10% of diameter movement. Of course, this scenario is hardly consistent with either the design for buckling (uniform radial pressure) or the potential assumption of tunnel load conditions that depend on stable soil arches.

To avoid this logical inconsistency, our industry employs another one and uses the F 1216 design method assuming that the host rigid pipe is replaced by native soil that produces uniform radial pressure around the liner. This in turn, justifies using the modified Luscher buckling equation of F 1216 (and AWWA M45). It is difficult for this author to see the relationship of this model to reality under any reasonable set of assumptions (McAlpine 1993). Flexible pipe design for limited deflection would suffer the same contradictions to design assumptions and logical inconsistencies. If we must assume the total loss of host pipe load capacity and the logical implication of loss of soil side support, a more reasonable phenomenological design method would be to use the liner load capacity producing 5% deflection in the parallel plate test (ASTM D 2412) for pipe stiffness. This test condition emulates the buried liner with no soil side support but vertical loads at the top and bottom.

## **9.0 Utah State University Test Results**

Figure 1.0 shows the results of testing at Utah State University (USU) on 750 mm (29.5 in ) cracked non-reinforced concrete pipe, both lined and unlined, in their large soil cell. Figure 1(a) gives the results for a 21 mm (0.83 in ) Cured-In-Place-Plastic (CIPP) liner (Watkins 1988) whose short-term modulus of elasticity was assumed to be 2,760 MPa (400 ksi). The cracked pipes were installed and the soil cell pre loaded to 69 kPa (10 psi) before the CIPP liner was installed. This pre loading produced about 2% ovality in the cracked pipe before installation of the liner. However, the deflections shown in Figure 1a do not include this pre load deflection. The soil in the cell was silty sand (Type SM) with a relative density of 75% or less that should produce a soil modulus  $E_s'$  of from 690 kPa (100 psi) to 2.76 MPa (400 psi). The pre loading would have produced some compaction of the soil resulting in a modulus value toward the high end of this range of values. After pre loading and installation of the liner, the load was increased in discrete increments and the deflections recorded. No strain gages were used to measure liner stress.

Figure 1b gives the results of similar tests (Watkins 1993) for a Grouted-In-Place (GIP) 675 mm (26.5 inch) PVC liner. This test differed from the CIPP test in two ways. First, there was very little pre loading of the soil cell before the liner was installed. Pre loading consisted of 75-150 mm (3-6 inch) of soil cover over the broken concrete pipes required to hold the broken pipes together and to contain annulus grout that migrated through the pipe cracks. These conditions would likely lead to a soil modulus in the lower portion of the 690 kPa (100 psi) to 2.76 MPa (400 psi) range. Secondly, strain

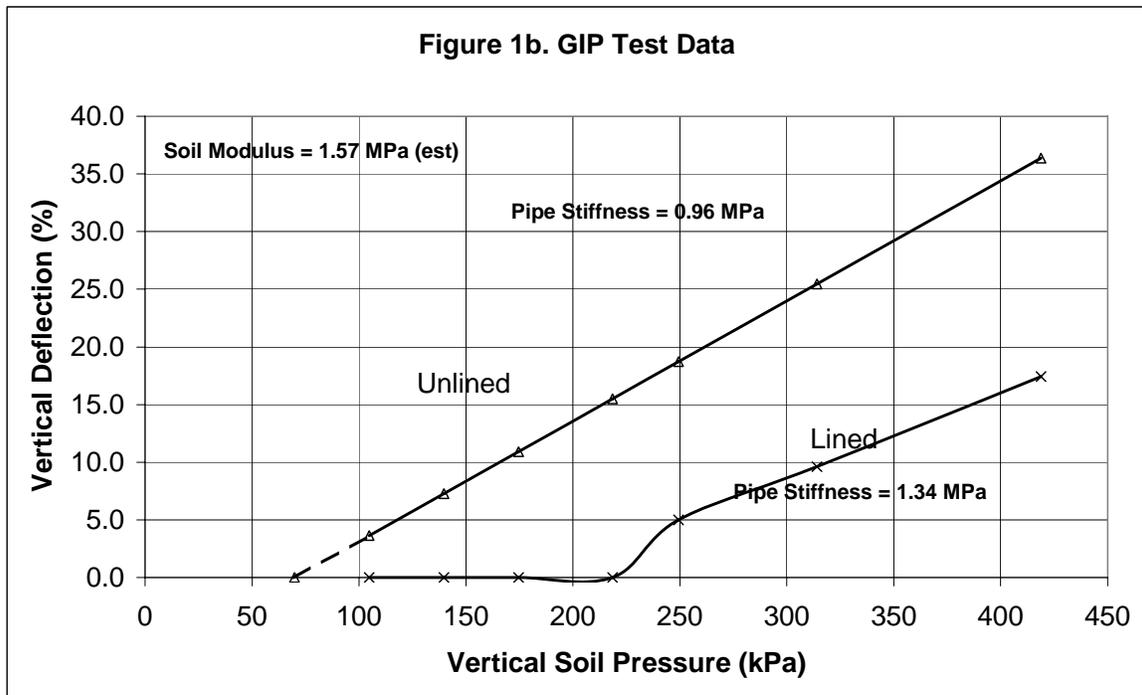
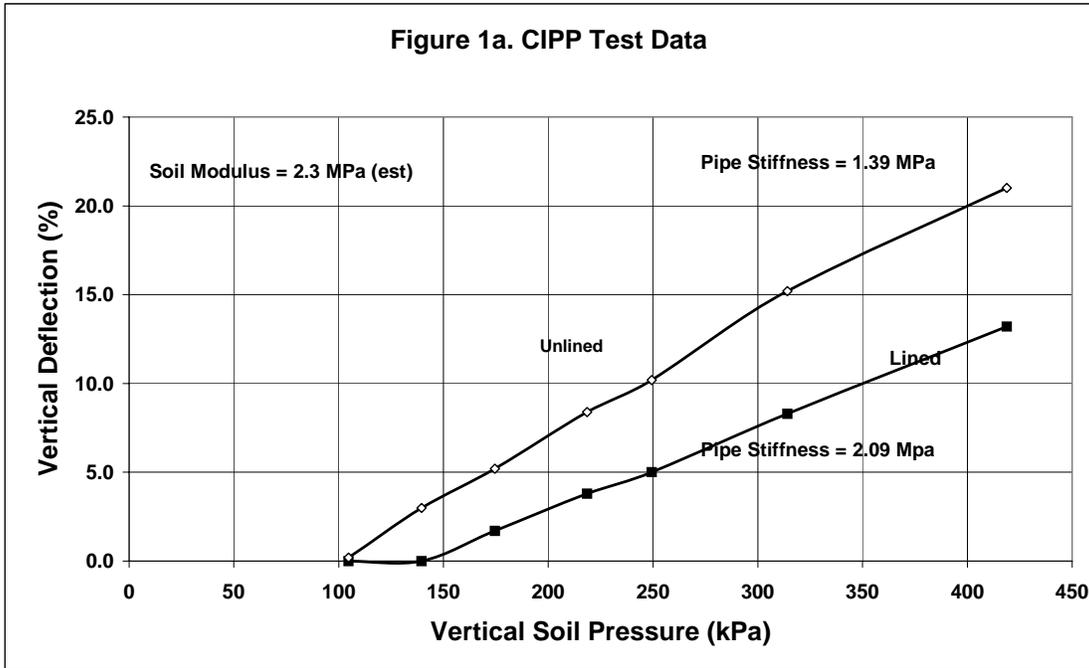
gages were employed at the crown, invert and springlines of the PVC liner. Deflections and strains were measured as the load was increased in discrete increments. In both tests the deflections reached about 20% and the duration of the tests was over parts of two consecutive days. The modulus of the PVC was about 2,760 MPa (400 ksi) while that of the grout was estimated at 24 GPa (3500 ksi). The moment of inertia equivalent thickness of the profiled PVC liner was 9.1 mm (0.36 inch).

The test results show two types of structural enhancements by the liners installed in an obviously “fully deteriorated” rigid pipe. The first is the increased (over that of the cracked concrete pipe) load required to initiate deflection of the lined pipe. This is referred to as strength enhancement. The second enhancement is the increase in pipe stiffness indicated by a decrease in the slope of the deflection versus load curves (fairly linear).

As the broken concrete pipes have essentially zero rigidity (resistance to initial measurable deflection), the enhancement is taken as the increase from the projected zero deflection intercept of the load-deflection line for the unlined pipe. This effect of the liners is due to bonding of the liner materials (liner itself for CIPP and grout for GIP) to the inner wall of the concrete pipe, thus increasing the rigidity of the composite structure over the unlined pipe. From a rehabilitation design standpoint, it is unlikely that the required bonding for strength enhancement can be reliably obtained in practice for any of the close-fit liners (CIPP and Fold and Formed). Further, strength enhancement due to the bonding of the CIPP liner to the concrete pipe is only of significant magnitude because of the state of the cracked pipe.

Due to large differences (about 10:1) in the modulus of concrete to that of most flexible liners, one can expect a much smaller strength increase of this same liner in an uncracked concrete pipe (e.g., in a D-load test). An earlier paper (Ahmad and McAlpine 1994), using a theoretical analysis and this test data for the GIP system, shows that the grout has effectively repaired the cracks and restored much of the pipe’s original (before cracking) rigidity.

Once the pipes begin to deflect under soil pressure, the ratio of slopes of the pressure-deflection graphs represents the greater ring stiffness of the lined pipe compared with the unlined pipe. Of course, the unlined pipe has no stiffness except that which is provided by backfill soil support. The two liners increased the stiffness of the cracked pipe by 50% (CIPP) and 40% (GIP). By differentiating the Spangler/Iowa deflection formula with respect to soil pressure and equating this to the measured value of slope of the unlined pipe graph, the soil modulus is estimated to be 2.3 MPa (330 psi) for the CIPP test and 1.6 MPa (230 psi) for the GIP test. Repeating this mathematical process for the lined pipe cases, and inserting the estimated soil modulus values, yields the pipe stiffness contributions of the liners to be about equal to the calculated or measured values of the flexural rigidities ( $E \times I$ ) of the respective liners. For the test durations employed in the soil cell tests, it is probably appropriate to use the short-term modulus of the liner materials. In rehabilitation design, however, the long-term modulus should be used.



**Figure 1 (a) & (b) – Load – % Deflection Curves from Utah State University Test**

Again, it is important to note the contribution of the soil to the load carrying capacity of the pipes. The more highly compacted soil of the CIPP test produced a slightly higher stiffness (lower slope of load-deflection graph of Figure 1) for the unlined pipe than the GIP lined pipe in the lower modulus soil. Further, it must be recognized that for both tests the stiffness of the cracked pipe-soil was about twice the stiffness contribution of the liners. For higher values of soil modulus the liner thickness must

increase (as the cube root of soil modulus) to maintain liner influence on combined pipe stiffness. It has been suggested (Schrock 1999) that minimum soil modulus value of 6.9 MPa (1 ksi) be used in rehabilitation liner design. Such a design for the CIPP liner under discussion would require a 44% increase in thickness (from 21 mm (0.82 in ) to over 30 mm (1.18 in )) to produce the same 50% increase in combined pipe stiffness as obtained in soil of 2.3 MPa (330 psi) modulus. Another 8 mm (0.31 in) of liner thickness should also be added to the design value to account for a 50% reduction in liner material modulus due to long-term creep. Adding a design safety factor of 2 would bring the total required thickness to 48 mm (1.9 in.)! This liner would have a pipe stiffness of about 1.2 MPa (170 psi); it would deflect 5% under 61-m (200-ft) of 1.54 kg/m<sup>3</sup> (120 lb/ft<sup>3</sup>) soil prism load! And by implication, the cracked concrete pipe-soil structure is twice as stiff as the liner. Clearly, if such assumption about the soil ( $E_s' = 6.9$  MPa (1 ksi)) is valid, there is no need to add to the stiffness of the fully cracked concrete pipe-soil structure. Further deflection of the cracked host pipe is highly unlikely in such stiff soils and, therefore, the liner is equally unlikely to experience any significant stress.

Under the above scenario, the only function of a rehabilitation liner is to prevent soil infiltration into the pipe through the cracks that would destroy the soil support that is creating the very stiff structure. Soil erosion into the pipe most likely occurs when ground water is present. Thus, a logical design would provide a liner buckling strength adequate to resist the hydraulic pressure of expected ground water levels. Moody and Whittle (1996) make much the same argument with one major exception. Their argument all but ignores the deteriorated host pipe, which, in fact, does contribute considerably to the structural stability of their “tunnel.” Further, as stated previously, unless the host pipe deflects after the liner is inserted the liner will carry none of the earth load. In stiff soils the host pipe-soil will have such high stiffness that any stiffness added by a reasonable thickness liner will be negligible. The presence of the host pipe cannot, and should not, be ignored unless it can be shown that by doing so is a “worst case.”

The USU test of the GIP liner clearly shows that its primary contribution is in the repair and enhancement of the host pipe’s rigidity. The rehabilitation design method for this type of rigid liner was given in an earlier paper (McAlpine 1997).

## **10.0 Water Research Centre (WRc) (UK) Test Results**

In the early 1980s the Water Research Centre (WRc) in England conducted two tests similar to the USU tests discussed above. The first (WRc 1982) was conducted in a soil cell using cracked clay (600 mm (23.6 in)) with CIPP, and GRP (Glassfiber-Reinforced-Plastic) grouted liners and cracked concrete (875 mm (34.4 in)) pipes with GRC (Glassfiber-Reinforced-Cement) grouted liners. The cracks in the host pipes were taped and the inner surface was coated with a compound to prevent any type of bonding of the grout or CIPP resin to the wall of the host pipe. Thus, it can be assumed that forces on the liners were due to deflection of the host pipe and physical contact with the liner materials. One test on the unlined cracked clay pipe was conducted in “Good” soil (compacted sand). All other tests used “Poor” soil (loose sand).

The advantage of “Good” over “Poor” soil was obvious and significant; deflections of the unlined cracked clay pipe of 0.01% and 2.3%, respectively, for a vertical pressure of 100 kPa (14.5 psi). Under the same load pressure in “Poor” soil, the 14 mm (0.55 in) CIPP liner in the cracked clay pipe had a deflection of 1.3% while two different GRP liners experienced 1.1% deflection. It is interesting to note that the pipe stiffness (not including the 50 mm (1.97 in) grout annulus) of one of the GRP liners was twice that of the other, yet deflections were the same. Further, the pipe stiffness of the CIPP liner lies between the two values for the GRP liners. It may be that the stiffness of the unbonded grout rings (estimated to be 100 times that of the stiffest GRP liner) is overwhelming the liner effects.

The performance of the GRC liners in cracked concrete pipes was influenced by the strength of the annulus grout. The GRC liner bonded to higher strength grout (12 MPa (1.74 ksi)) had a deflection of 0.08% while the deflection of the GRC liner with the lower strength grout (8 MPa (1.16 ksi)) was 0.16%. The unlined cracked concrete pipe deflected 0.56% under the same vertical pressure of 100 kPa (14.5 psi) in “Poor” soil. The data given in this summary report (WRc 1982) is very limited and only discusses test results for very small values of loads and deflections. This fact makes it difficult to compare these results with those from the USU tests that covered a much larger range of loads and deflections and produced little data in the small deflection range.

Because of the lack of bonding to the pipe wall and the grout not bonding to the GRP liner, all three liners meet the WRc definition of Type II (flexible) liners that are designed to meet deflection limits and buckling under soil pressure (AWWA M45) and water pressure. None of the test data from these tests appear relevant to soil pressure buckling.

The second set of tests (WRc 1983) employed an abandoned 120 years old 600 x 900 mm (23.6 x 35.4 in) (nominal) egg-shaped, single ring brick sewer. Mortar loss was medium, being most evident in the crown, and there were a few displaced or missing bricks. The sewer was not substantially deformed with an average decrease of 50 mm (1.97 in) in the assumed original crown to invert dimension in test sections 1 – 7. Deformation was notably less in test sections 8 and 9 where the soil support was improved. Soil cover was removed to within 1.0 m (39.37 in) of the top of the sewer and a movable loading platform installed to allow variable loading over each of the 9 separate test sections. The types of liners tested were the same as previously described (WRc 1982) except that a second CIPP test was conducted with a grouted annulus. The nine test sections were: 1) 12 mm (0.47 in) CIPP with grouted annulus, 2) repointed brickwork, 3) original brick sewer, 4) 12 mm (0.47 in) CIPP, 5) glass reinforced cement/GRC, 6) polyester resin concrete/PRC, 7) low stiffness glass reinforced plastic/GRP, 8) medium stiffness GRP, and 9) high stiffness GRP. Except sections 2, 3, and 4, all liners were grouted with the same 8.5 MPa (1.23 ksi) compressive strength (nominal) grout. However, due to the “off-the-shelf” standard sizes of most of the liners the annulus sizes varied considerably. Also there was considerable variation in the 28-day strength, ranging from a low of 5 MPa (0.73 ksi) to 11 MPa (1.6 ksi). No attempt was made to prevent grout bonding to the brick pipe interior.

Strains at the crown and “springings” and vertical and horizontal deflections of each test section were measured as the sewer was loaded to the maximum possible, subject to physical constraints, and repeated five times or until failure whichever came first. The maximum capacity of the loading system was about 0.8 MPa (0.12 ksi). After load testing, the elevation of the invert was measured and in all cases no movement of the structure was detected.

The relevant principal conclusions of this report (WRc 1983) are:

1. All lining systems tested, except the repointed brick and the un-grouted CIPP, produced significant increases in the short-term load bearing capacity of the sewer.
2. Grouting was essential to consolidating the brick structure and significant improvements to its load capacity.
3. The grouted liners formed very rigid structures acting as arches as evidenced by small and nearly equal horizontal and vertical deflections (< 2 mm (0.078 in)) with tensile strains at the crown and compression at the “springings”.
4. Cores taken indicate that the shear bond between the liners and the grout essential for composite structural behavior is very small. Test data show composite action for sections 5 (GRC), 6 (PRC) and 9 (high stiffness GRP). Sections 1 (grouted CIPP), 7 (low stiffness GRP) and 8 (medium stiffness GRP) showed no bonding to the grout or composite action.
5. Section 1 (grouted CIPP) produced a very rigid structure but the liner did not bond to the grout. By WRc definition, this is a Type II (flexible structure) liner. However, Appendix 4 (WRc 1983) attempts back calculating soil modulus using the data from tests of section 1 but the calculated values are far larger than any reasonable soil modulus value (45 to 388 MPa (6.53 to 56.3 ksi)). Because the CIPP liner stiffness term is less than 0.2% of the soil modulus term in the deflection equation, this calculation is probably an improper use of Spangler’s deflection equation. On the other hand, these very high, calculated values of soil modulus could be indicating the high stiffness of the soil-brick-grout composite structure.

These tests produced no data that would, in any way, substantiate or support the soil pressure buckling design method of the “fully deteriorated” pipe condition. In fact, the performance of the un-grouted CIPP (section 4) demonstrates that a flexible liner may not be capable of preventing failure (collapse) in a “fully deteriorated” brick sewer. (The brickwork failed in shear at the crown and concentrated the load on the liner. It failed at approximately the same load as the original, unlined sewer.)

## 11.0 Pipe Rehabilitation Council Symposia

The Pipe Rehabilitation Council (PRC) has held four separate symposia on the subject of appropriate design for the “fully deteriorated” pipe condition. The first symposium was held in June 1997, just two months after Schrock and Gumbel presented their paper at NO-DIG '97. Three more have been held at various venues during the semi annual meetings of the ASTM. While the first symposium was conducted as an open forum amongst the invitees, the other three have had invited speakers or panelist. A free exchange of ideas and data was encouraged and written comments were collected and distributed to all attendees. Attendance ran 20 to 25 per symposium.

The following is a highly condensed summary of the discussions at these symposia. Readers should be aware of the dangers of oversimplification of complex technical issues and not be misled by condensation of important (and sometime complex) technical details.

1. Consensus is that, in general, Schrock & Gumbel have a valid thesis concerning the shortcomings of the current design practice.
2. Current use of the term "fully deteriorated" is inappropriate and illogical.
3. Current design equations for both "partially deteriorated" and "fully deteriorated" conditions are flawed.
4. Soil loadings currently used in rehabilitation design are probably overly conservative; however, we need to consider future possible changes in total loads and in pressure distributions (e.g., future excavations near the rehabilitated pipe).
5. Better models and design equations are available but some work is required to validate these to insure that these potentially new industry standards don't suffer from their own limitations and shortcomings.
6. The behavior of the host pipe and the liner should be studied together in a finite-element analysis of the soil structure interaction.
7. The deflection of the liner is not independent of that of the deteriorating host pipe. The buckling resistance of a liner inside a deteriorated rigid pipe with large deflection is not similar to that of a tunnel liner.
8. Calculations are needed of (i) the deformations and stresses in the liner, and (ii) the instability of the liner resulting from the deformation of the deteriorating host pipe when the liner is subjected to hydrostatic pressure while the deformations are constrained by the host pipe. This requires a finite-element model of the soil-structure interaction. The results should then be verified experimentally. This experimental verification may be difficult and expensive.

## 12.0 Conclusions

This part has examined results of test performed on lined and unlined cracked rigid pipes supported by soil (soil cell tests) and one set of test on an in situ brick egg shaped sewer with several rehabilitation liners. None of the data examined supports a design model based on the Luscher/AWWA M45 soil buckling. In fact, the tests in the abandoned egg shaped brick sewer lined with an un-grouted CIPP liner showed that the

flexible liner had no beneficial affect on liner failure (WRc 1983). Further, the shear failure at the brickwork crown produced a concentrated load on the liner that is clearly inconsistent with uniform radial pressure required in the buckling model. In all circular pipe cases the test data show that cracked rigid pipes have significant load carrying capacity as a flexible structure, the magnitude of their capacities depending largely on the quality of the supporting soil. The USU tests showed that flexible liners could increase the stiffness of a cracked concrete pipe. However, the liner thickness required to produce a given percentage increase in stiffness increases with increases in soil modulus. Exactly the opposite is true of the Luscher soil buckling design, i.e., liner thickness decreases as the soil modulus increases. In very stiff soils the effective stiffness of the cracked rigid pipe-soil structure is very high and reasonably thick flexible liners would do little to enhance the structure's stiffness. If the cracked concrete pipe has good soil support, the only reason to have a flexible liner is to preserve the soil support by preventing soil washing into the pipe and creating soil voids. Under this scenario the liner carries none of the earth load and may be designed for hydrostatic buckling only. If the cracked pipe has little or no soil support, a flexible liner will be of little use. And certainly the Luscher design that assumes uniform radial soil pressure would be inappropriate.

Any industry wide design standard must be appropriate for, at least, major areas of application. Clearly, the lining of deteriorated concrete pipe is a major application for the rehabilitation industry. As pointed out by Serpente (1994), the most likely failure mode for concrete pipe is cracking. Thus, a design method must be found for liners in cracked concrete pipe; the current Luscher model is not appropriate. However, if the soil support is present and dependable, the current design using prism loads may be conservative, depending on the water head present and the value of soil modulus used.

The argument that tunnel loading should not be used because future changes in soil support may lead to failures mistakenly assumes that the current design does not suffer this problem. No pipe, rigid or flexible, designed assuming good soil support can perform if that soil condition is either not achieved or deteriorates badly for any reason. The only "safe" design of flexible liners would be to assume no soil support, use prism loads and design thickness to achieve the required pipe stiffness. In the vast majority of cases this would be seriously over conservative and costly. The answer to this dilemma is to make realistic assumptions, based on good geotechnical data, design appropriately and stop depending on logically flawed design assumptions. Further, more research is needed on liner-cracked rigid pipe-soil interactions, deformations and stress resultants.

Further study and discussion of the topic of this part will be held by the interest groups referenced in this topic. Readers who wish to contribute to this technical dialog should contact this author or Jay Schrock, chairman of the ASCE-PLD PINS task committee.

## PART III

### NEW APPROACH TO DESIGN OF CIRCULAR LINER PIPE TO RESIST EXTERNAL HYDROSTATIC PRESSURE<sup>2</sup>

#### 13.0 Introduction

Current methods for structural design of flexible gravity sewer liners, as typified in North America by the Appendix X1 to ASTM F1216 (1998), generally consider two external load cases for dimensioning of the liner pipe.

The first is sustained hydrostatic pressure due to groundwater acting in the annular space between the liner pipe and the host pipe. Since gravity sewers in need of renovation invariably leak, this load case applies regardless of host pipe condition or whether the aim of the rehabilitation is primarily structural or simply to provide a barrier against internal corrosion or ex-filtration. Even where the permanent groundwater table is below pipe invert level, the liner must generally be designed to resist a short-term hydrostatic head, which could arise under storm conditions.

The second load case supposes that earth and traffic loads will in due course be transferred from the existing pipe-soil structure to the liner pipe. The likelihood of this occurring is assumed to be a function of the condition of the sewer at the time of lining. In the great majority of practical situations, however, little or no such load transfer ever takes place because the lining process effectively and permanently locks in the existing equilibrium of even quite badly deteriorated sewer structures. So far this fact is recognized explicitly only in the UK design procedure (WRc/WAA, 1994) and with few exceptions the treatment of the soil load transfer case in other national methods is irrationally over-conservative (Gumbel, 1998).

This excessive conservatism is particularly acute in the treatment of the “fully deteriorated pipe condition” by ASTM F1216, for two reasons. First the soil load reaching the liner pipe is overestimated by treating the liner as though it had been directly buried, whereas the more correct analogy in circumstances where the existing structure may continue to deteriorate after renovation is with the situation of a tunnel lining (Schrock & Gumbel, 1997). Secondly, the formula used to describe liner response to transferred soil load, and hence calculate the required wall thickness, has been incorrectly modified from an already conservative theory for direct-bury applications in a way, which entails further irrational safety factors (Gumbel, 1998).

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Recognition of the unnecessary burden imposed on the North American industry by such conservative procedures led in April 1999 to the formation of a new Rehabilitation Design Task Group of the ASCE Pipeline Infrastructure (PINS) committee, of which the writer is a member. The goal of the group is to prepare an updated design procedure for circular sewer liners of all types, with particular emphasis on a more realistic assessment of the conditions for transfer of soil load, and of its true extent and impact in such cases. Since the adjustments made are expected to result in the hydrostatic load case proving the critical determinant of liner wall thickness or ring stiffness in most practical situations, it was considered a timely opportunity also to update the ASTM design procedure for the related limited state of restrained creep buckling.

It is in this context that a synthesis of recent research into mechanisms of restrained hydrostatic buckling, leading to a proposed new approach to the design of nominally circular, flexible sewer liners, is now offered.

#### **14.0 Origins and shortcomings of the current ASTM formula for hydrostatic buckling**

The formula for hydrostatic buckling of liner pipe still used in most rehabilitation design practice worldwide originated in the UK almost 20 years ago. It is based on the theoretical hydrostatic buckling pressure of a long, perfectly circular, unrestrained pipe given by Eq. 1.

$$(P_{cr})_0 = 24 \cdot E^* \cdot I / D^3 = 2 E^* \cdot (t/D)^3 \quad (\text{Eq.1})$$

where:  $E^* = E/(1 - \nu^2)$  = plane strain modulus

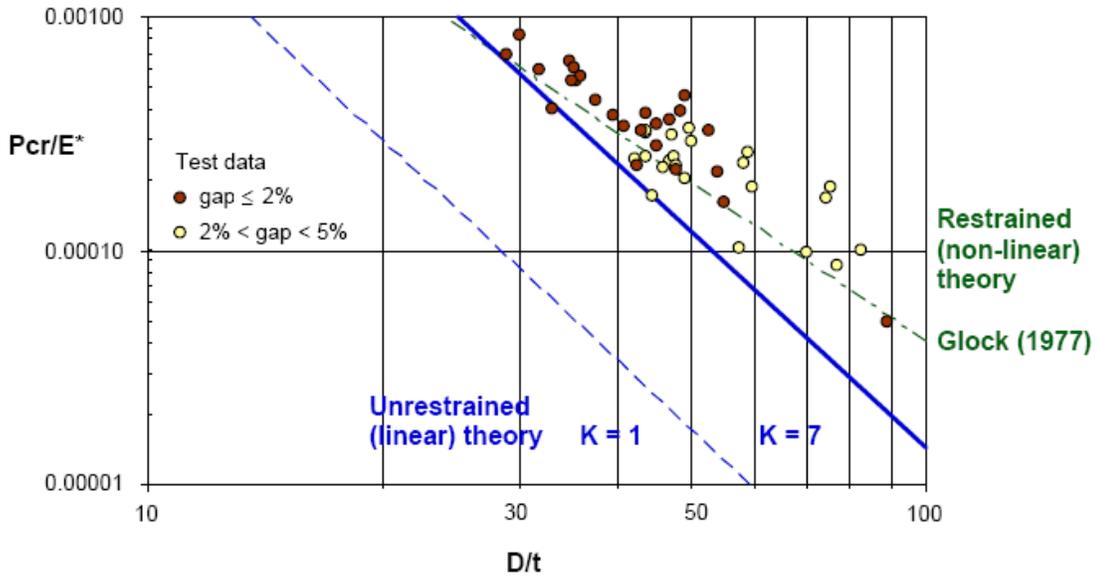
$E$  = flexural modulus (short or long-term, as appropriate)

$\nu$  = Poisson's ratio

$I$  = second moment of area (=  $t^3/12$  for solid-wall pipe of thickness  $t$ )

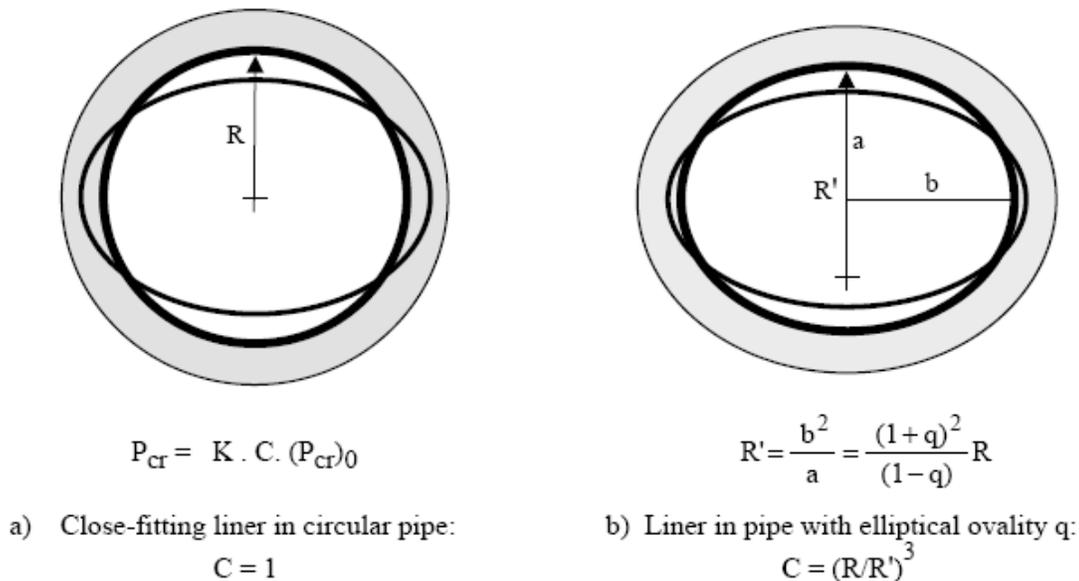
$D$  = median diameter of pipe ring (= outside diameter minus thickness)

To account for the observed enhancement of this unrestrained buckling pressure in the first experiments in which cured-in-place pipe (CIPP) liners inserted in steel casings were subjected to external water pressure, Aggarwal and Cooper (1984) defined a simple empirical factor  $K$  applied to Eq. 1 for design purposes. As illustrated in Figure 2, they found that a value of enhancement factor  $K = 7$  provided a close lower bound to the experimental data, and this was the figure subsequently adopted for CIPP in the first WRc Manual, and which found its way in due course into ASTM F1216.



**Figure 2** – Aggarwal’s 1984 hydrostatic buckling data compared with contemporary theories

Based on Figure 2, this simple design procedure is clearly conservative, especially given that the experimental liner pipes were inserted into the steel test casing after cure. This resulted in much larger annular gaps (up to 5%) and hence less radial restraint than would occur in the field where such liners are truly “cured in place”. But the approach is nevertheless essentially empirical, and so limited in validity to the particular CIPP product and range of dimension ratio  $D/t$  tested. The first of these points was borne out in a major US testing program in which various different CIPP products were found to exhibit quite different experimental  $K$ -factors (Guice et al., 1994).



**Figure 3** – Liner deformations implicit in current ASTM design formula based on linear theory of unrestrained buckling with enhancement factor  $K$  and ovality factor  $C$

There is also at least one major theoretical inconsistency in the simple approach. As illustrated in Figure 3a, the linear buckling mode assumed involves liner deformations breaching the boundary conditions imposed by the restraining host pipe. Although there was an alternative, non-linear theory consistent with the boundary conditions available at the time that Aggarwal was analyzing his test results (Glock, 1977), this was evidently missed, probably because it was published only in the German language literature. The unmodified Glock theory, also plotted on Figure 2, follows the trend of the data better and with a shallower slope, suggesting that the mean K-factor increases with increasing  $D/t$ . Neither theory however takes explicit account of initial gap imperfections or explains the large, apparently random scatter of Aggarwal's data.

The WRc and ASTM formulae did nevertheless from the outset take explicit account of the initial out-of-roundness or “ovality” due to vertical deformation of the sewer pipe prior to lining. The full ASTM formula for buckling pressure expressed in the notation of this part (without safety factor) is:

$$P_{cr} = 2 \cdot K \cdot C \cdot E^* (t/D)^3 \quad (\text{Eq. 2})$$

where  $C$  is a semi-intuitive “ovality factor” derived by substituting for radius  $R = D/2$  in the buckling formula for unrestrained circular liner pipe the local maximum radius of curvature  $R'$  of an elliptically deformed host (see Figure 3b). It is only relatively recently that experimental data have become available to test the validity of this ovality factor, and the results published by Boot & Welch (1996) suggest that the ASTM  $C$ -factor, like the rest of the formula, is conservative (see Figure 4). But closer examination of these and other test data still being evaluated (Seeman et al., 2001), as well as broader theoretical considerations (Boot & Gumbel, 1998), cast doubt on the validity of attempting to isolate the impact of ovality as an independent factor on buckling pressure. The need for an integrated approach to predict the combined effect of ovality, gap and other imperfections provides a further imperative for updating the current theoretical model.

## 15.0 Mechanisms of restrained hydrostatic buckling

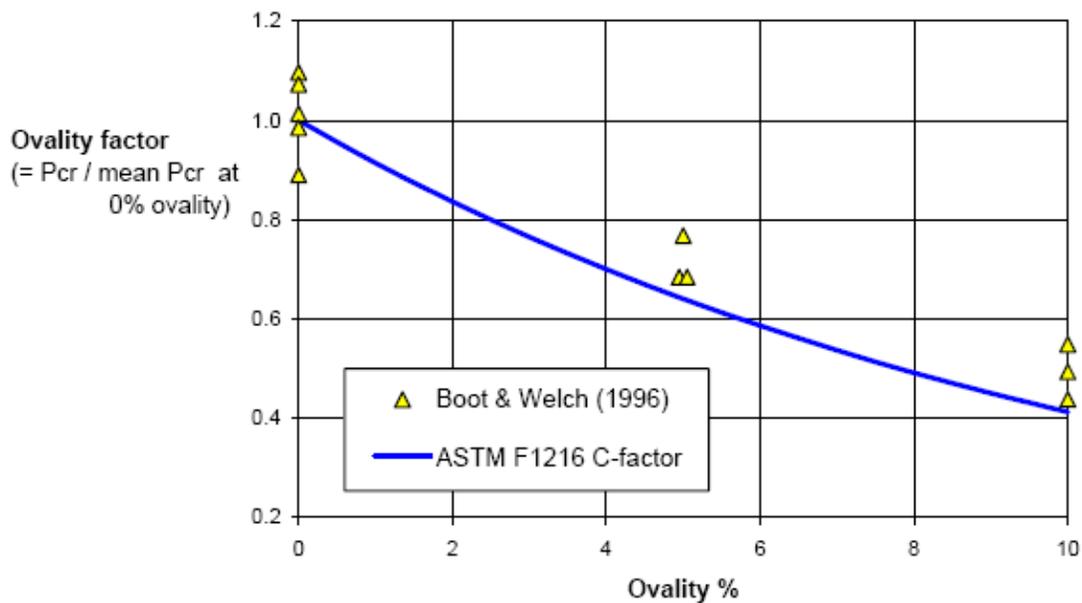
Figure 4 illustrates the steps leading to buckling failure of an encased circular liner pipe subject to external hydrostatic pressure, as observed in numerous laboratory experiments in the US and UK in recent years (e.g. Guice et al., 1994; Boot & Welch, 1996; Boot & Javadi, 1998a).

If the unloaded liner is a tight fit to the host pipe, it can initially deform only by uniform hoop compression (Step 1). The resulting slack in the system allows the liner to lift away from the host pipe in either an asymmetric (one-lobe) or symmetric (two-lobe) mode (Step 2). As pressure is further increased the inward deformation of the lifted lobe or lobes is accompanied by a shortening of lobe length and increase in ring compressive strain. Eventual snap-through (Step 3) is in essence a form of geometric instability, in which the buckling pressure is associated with the critical lobe length at failure. The

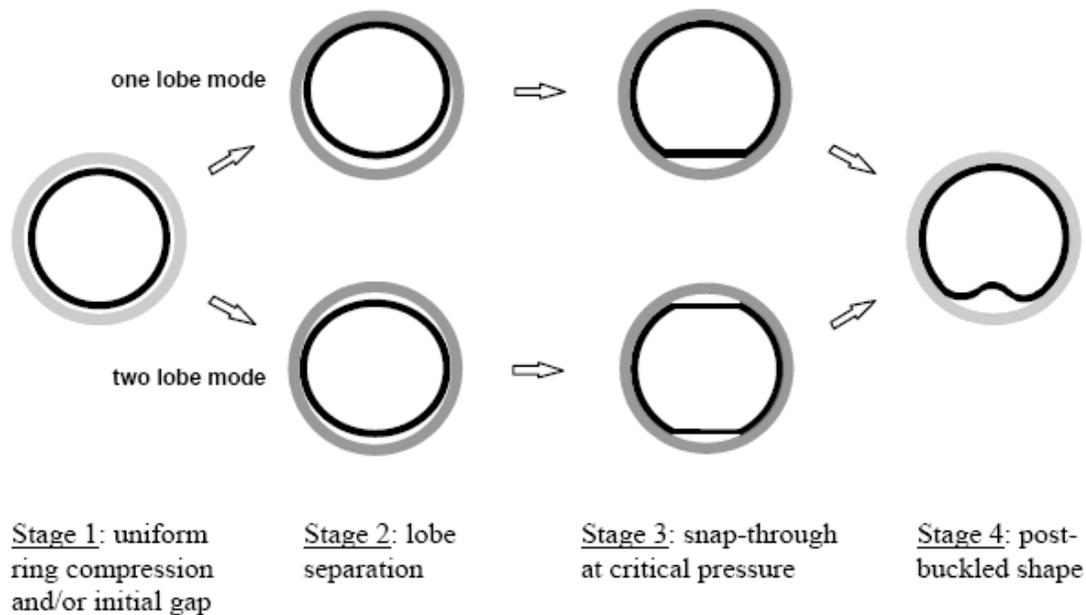
critical lobe length increases as a result of initial annular gap and both elastic and creep compressive strains contributing to further reduction in liner perimeter under pressure. This controlling influence of hoop compressive strains is what distinguishes the buckling mode as essentially non-linear.

The pre-buckling deformations leading to instability are not generally discernible with the naked eye, and regardless of whether one or two lobes develop, snap-through can only occur at a single point because the associated release of strain energy instantly stabilizes any other incipient points of failure. The resulting manifestation of post-buckling deformation (Step 4) as apparently single lobe should not therefore be confused with the buckling mode itself.

The original non-linear theory developed by Glock (1977), as plotted in Figure 2, assumed a single lobe mode associated with initially perfect circular geometry. However as soon as some initial gap is introduced, the liner pipe will tend to deform elliptically while taking up the slack as if unrestrained (Figure 3a), and then be locked into the symmetrical two-lobe mode up to failure. For nominally close-fitting liners, this tendency has been confirmed in the great majority of cases where pre-buckling deformation measurements have been made. This is contrary to the expectation expressed by Moore (1998) that a relatively loose-fitting liner tested horizontally and pressurized by water would tend to float and create an asymmetrical initial gap favoring the one-lobe mode. In the additional presence of even modest initial ovality (Figure 3b), for example the typical 2% tolerance on diameter associated with manufacture of new concrete and clay pipe, the liner will be even more strongly predisposed to follow a symmetrical deformation mode. Two-lobe deformation has indeed been observed in 100% of appropriately monitored buckling experiments in horizontally aligned, ovalized casings to date (Seeman et al., 2001).



**Figure 4** – Experimental ovality factors for liner of  $D/t = 45$  compared with ASTM C-factor



**Figure 5** – Steps in non-linear hydrostatic buckling of encased circular liner pipe  
(Liner deformations fully consistent with boundary conditions)

In view of their important influence on the qualitative as well as quantitative buckling response of restrained liner pipe, it is clear that any fully consistent design theory must take explicit account of imperfections.

## 16.0 Development and testing of a fully consistent design theory

Boot (1998) extended Glock's closed-form theory to take account of initial gap imperfections and also to reflect the observed two-lobe buckling mode. The generalized buckling equation he derived, when converted to plane strain conditions for long pipe, can be expressed in convenient dimensionless form simply as:

$$P_{cr} / E^* = c.(D/t)^m = c.(t/D)^{-m} \quad (\text{Eq. 3})$$

where both the coefficient  $c$  and the power  $m$  of  $D/t$  are functions of imperfections in the system.

For zero imperfection  $m = -2.2$ , and the coefficient  $c$  has value of 1.003 for one-lobe buckling (corresponding to Glock's published solution) and 1.323 for two-lobe buckling. As the mean gap/radius ratio  $w_{02}/R$  increases, both  $m$  and  $c$  increase, and at very large gaps approach 2-lobe values of -3 and 2 respectively, corresponding to the unrestrained buckling of Eq.1.

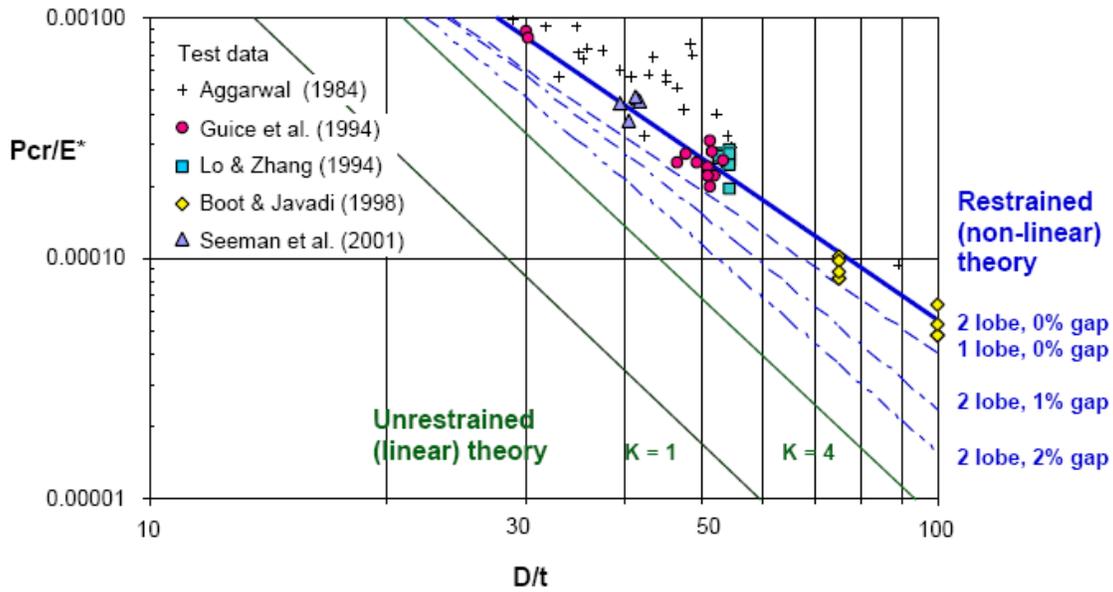
These basic predictions of the Boot-Glock theory are plotted in Figure 6 and compared with experimental data from a variety of sources covering a wide range of  $D/t$  values from 30 to 100. The top two theoretical lines correspond to the “perfect” two and one-lobe solutions respectively, separated by a constant factor of 1.32 independent of  $D/t$ , and hence parallel on the log-log plot. The next two lines down show how introduction of symmetrical gap imperfections  $w_{02}/R = 1\%$ , and then  $2\%$ , not only reduces the 2-lobe buckling pressure but also progressively changes the mean slope of the curve closer to that of the unrestrained theory.

Although considered by Boot to be near enough straight, the lines for non-zero gap in fact curve downwards slightly, implying some variation of  $m$  with  $D/t$ . As previously pointed out by the writer (Gumbel, 1993) this apparent increase in sensitivity to relative gap  $w_{02}/R$  (or in the case of one-lobe buckling  $w_{01}/D$ ) as dimension ratio  $D/t$  increases can be attributed almost entirely to the relationship of gap size to liner thickness<sup>3</sup>. Figure 7 shows that the theoretical reduction factor on buckling pressure due to gap alone hardly varies with  $D/t$  if expressed as a function of  $w_{02}/R$ . This linkage to thickness can best be understood in terms of the geometry of the buckle lobe as snap-through is approached (Step 3 in Figure 5). Although the increase in critical lobe length due to initial slack in the system is a function of gap expressed as an imperfection on diameter or radius, the associated reduction in snap-through pressure arises from the increase in slenderness or length/thickness ratio of the buckle lobe.

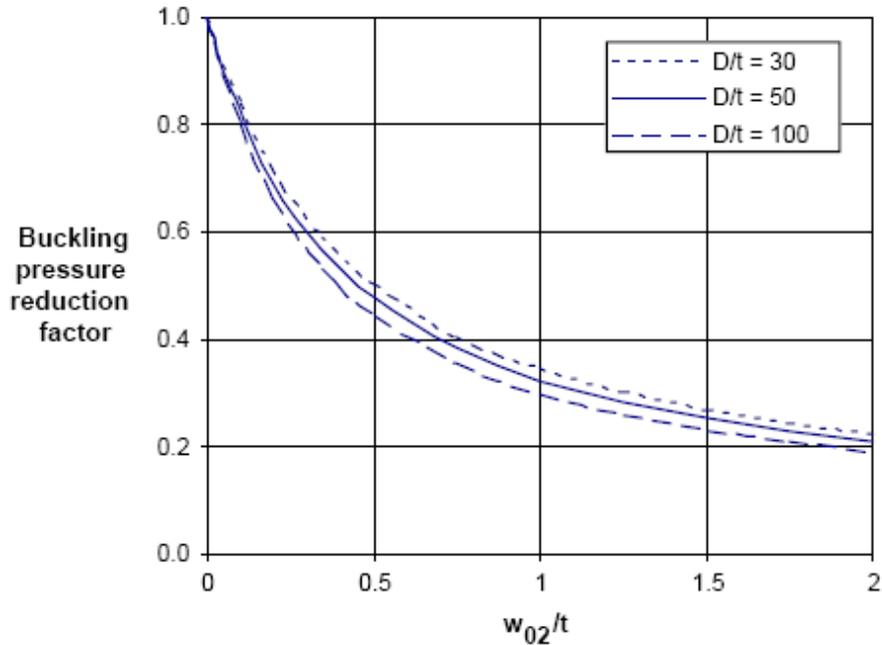
To facilitate comparison with the theory, all the experimental buckling data plotted on Figure 6 have first been corrected to perfect close-fit conditions by dividing the measured buckling pressure in each case by the reduction factor derived via Figure 7 from the mean measured gap imperfection and liner thickness. All the data plotted are for CIPP liners either cast directly into circular casings, or subsequently inserted with  $w_{02}/R \leq 2\%$ . The only exception is the set of points from Guice et al. (1994) plotted at  $D/t$  of around 30, which refer to a folded PVC product.

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<sup>3</sup> See Boot (1998) for notation: for the same relative size of liner to host pipe,  $w_{01} = 2 w_{02}$



**Figure 6** – Various gap-corrected hydrostatic buckling data compared with current theories for perfectly close-fitting, perfectly circular liners



**Figure 7** – Reduction factors on two-lobe buckling pressure due to symmetrical gap imperfection expressed as a function of gap/thickness ratio

Apart from the Aggarwal data, which due to deficiencies in the early test rig design discussed by Moore (1998) exhibit exceptional scatter skewed to the high side, the gap-corrected test results plotted in Figure 6 clearly follow the slope predicted by the

non-linear Boot-Glock theory. Furthermore the data are centered on the 2-lobe theoretical line, with a scatter band everywhere within 20%, at least on the low side. Given the high imperfection sensitivity of non-linear buckling at near-to-perfect circular geometry, as implied by the steep initial slope of the reduction factor curves in Figure 7, this provides strong validation of the 2-lobe theory as a basis for practical design of the CIPP and folded PVC products tested.

It is important that readers are not tempted to conclude, as some researchers have done, that it is appropriate to design such liners assuming a one-lobe buckling mode merely because the theoretical one-lobe curve in Figure 6 provides a convenient, if somewhat wide, lower bound to the experimental data. This would be irrational for two reasons:

- i) Although a small proportion of liners in near perfect circular casings have exhibited one-lobe buckling in laboratory tests, for field conditions it is normally necessary to design for at least 2% ovality imperfection, which invariably produces two-lobe buckling;
- ii) The loss of consistency between theory and observed buckling modes could lead to errors in quantifying relevant imperfections and predicting their effects.

This does not rule out one-lobe buckling modes being relevant to other lining techniques with different characteristic imperfections. The great advantage of a consistent theory over the current empirical approach is its ability to reliably predict the behavior of structures of types and in dimension ranges beyond testing experience. Much of the advantage of introducing non-linear buckling theory is however lost if, as in the latest German recommendations (ATV, 2000), a liner design methodology is built unquestioningly on the original one-lobe Glock theory.

## **17.0 Classification and combination of imperfections**

Most of the literature agrees on the need in principle to consider three distinct types of geometric imperfection of liner pipes. In addition to the gap and ovality imperfections already discussed, a “wavy” imperfection of the liner pipe wall in the shape of an incipient buckle lobe can, in theory, significantly diminish external pressure resistance (El Sawy & Moore, 1997). The common description of such wavy intrusions as “local” imperfections is however potentially misleading in the present context, because in order to have an effect on the cross-sectional response of a restrained liner pipe they need to be continuous for a few diameters along the pipe axis. Any truly localized imperfection of the existing sewer pipe wall that is transferred to a close-fitting liner, e.g. a displaced brick or broken pipe segment, will in practice produce a three-dimensional stiffening effect. The preferred term used here therefore is longitudinal imperfection.

Typically in the stability analysis of structures a wavy “eigenvalue” imperfection is the most critical that can be imposed, but the theoretical impact of a longitudinal imperfection of height  $\Delta$  in restrained liner pipe proves to be slightly less severe than that of gap. Moore (1998) has shown that, as in the case of gap imperfections illustrated in

Figure 7, the buckling pressure reduction is a function primarily of  $\Delta/t$ , and insensitive to  $D/t$ . But whereas a gap equal to the liner thickness will reduce the buckling pressure by just over one half, at  $\Delta/t = 1$  the pressure is not quite halved. These observations only apply, however, when considering reductions from the theoretical buckling pressure of an otherwise perfectly circular and close-fitting liner. As soon as different types of imperfection are combined, as invariably happens in practice, the high imperfection sensitivity of the perfect structure, represented by the steep initial slope of the plots in Figure 7, is progressively diminished.

For the analysis of combined imperfection effects it is helpful to define and distinguish between the characteristic imperfections associated with a particular renovation technique, and the system imperfections due to irregularities in the host pipe being lined (Boot et al., 2001). Examples of these two categories of imperfection are illustrated in Figure 8.

Looking down column a) of Figure 8, the most important characteristic imperfection of CIPP liners, as well as the close-fit PVC product analyzed in Figure 6, is shrinkage gap. In grouted slip-lined systems, this gap may be largely (although, due to heat of hydration of the grout, not entirely) eliminated, but is replaced by a characteristic ovality due to manufacturing tolerances, bending during insertion and/or deformation under grout pressure.

The example of characteristic longitudinal imperfection illustrated is the incipient buckle along the original fold line of certain close-fit thermoplastic pipes. This is in fact more of a material imperfection than a purely geometric one; it is due largely to residual stresses in the liner arising from the processes of folding and reversion.

The resulting impact on buckle location and critical pressure in folded PE pipes has been reported by Glanert (1992) and Alberding & Falter (1997). Other folded pipe materials and processes are not necessarily affected: for example the reduction factor on buckling pressure of the folded PVC product tested by Guice et al. (1994) and analyzed in Figure 6 is fully accounted for by the measured gap imperfection.

Other potential characteristic imperfections, not illustrated, are localized variations in material properties or thickness around the circumference of the liner. These have been analyzed by Boot (1998) but found to have only very minor effect compared with gap or ovality. For design purposes the key is to identify the dominant imperfections of each material and process. Moving to column (b) of Figure 8, the dominant system imperfection of deteriorated sewers affecting close-fitting liners of all types is ovality. This is not however commonly the elliptical ovality associated with deformation of flexible pipes: in cracked rigid pipes the typical “4-hinge” deformation creates a more severe imperfection for the same relative reduction in vertical diameter.

True longitudinal system imperfections are relatively uncommon: a layer of sediment encrusted in the pipe invert is one possible example, but usually removed by cleaning. Provided a conservative value of ovality is assumed, it should not normally be

necessary to include any default value of longitudinal system imperfection for design. System-related gap imperfections (not illustrated) may also arise where a circular slip-lined or close-fit pipe is limited in diameter by constrictions such as locally high ovality or offset joints. Such annular gaps would however normally be grouted.

For each particular renovation technique and host pipe to be lined it is necessary to consider the combined effect on buckling pressure of all the relevant imperfections. Due to the progressive reduction in imperfection sensitivity as total imperfection increases, the application of separate gap, ovality and waviness reduction factors, as advocated by the new German design guide (ATV, 2000), is unduly conservative. The real challenge for practical implementation of the new theory is to find a way to present it in a transparent and easily applied format, which nevertheless preserves the principle of consistency.

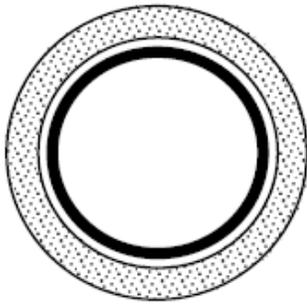
## **18.0 Practical applications of the new design approach**

The basic design concept proposed is to develop buckling pressure or safe water head charts for each renovation technique which already incorporate its characteristic imperfections, but allow variation of the relevant system imperfections according to the condition of the sewer to be lined. The most appropriate form of chart may vary according to the type of renovation technique and/or liner material: three examples are considered here.

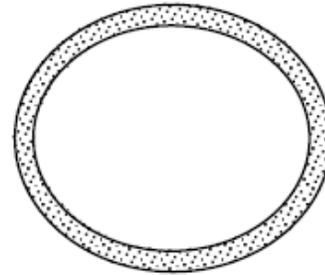
### ***18.1 Cured-in-place pipes of homogeneous wall construction***

Figure 9 shows a typical design chart appropriate for CIPP.  $P_{cr}/E^*$  versus  $D/t$  curves are presented for different system ovalities which also incorporate a characteristic gap imperfection, in this case 0.5%. Since a closed-form solution for computing the combined effects of gap and ovality has yet to be developed (Boot et al, 2001), the curves were based on the results of finite element analysis by Toropova & Boot (1998). The “4-hinge” form of ovality appropriate to rigid pipe renovation (Figure 8b) has been assumed. For this part only, the corresponding curves implied by ASTM F1216 are included for comparison.

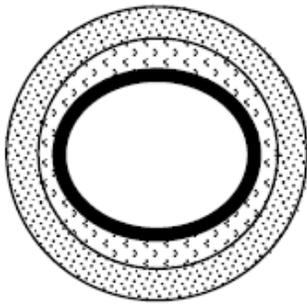
It may be observed that the combined “2% ovality + 0.5% gap” curve is higher and of shallower mean slope than the pure 2-lobe, 2% gap curve plotted in Figure 6. This is because the effect on buckling pressure of a given percentage ovality on its own is quantitatively much less than that of the same amount of gap, and also qualitatively different. As already illustrated by Figure 4 in contrast to Figure 7, the pressure reduction due to ovality develops more gradually than that of gap, without the steep initial drop indicative of high imperfection sensitivity.



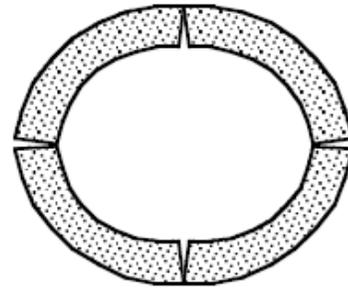
**Gap** (e.g. due to thermal shrinkage of liner)



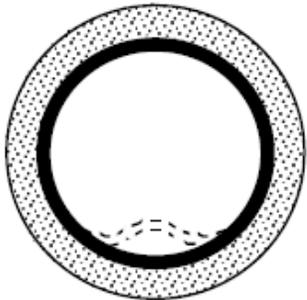
**Ovality — elliptical**  
(deformed flexible pipe)



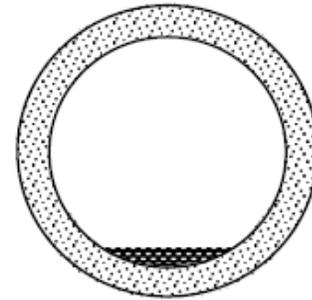
**Ovality** (e.g. deformation of grouted slip-lined pipe)



**Ovality — 4-hinge**  
(deformed rigid pipe)



**Longitudinal** (e.g. original fold line of close-fit pipe)

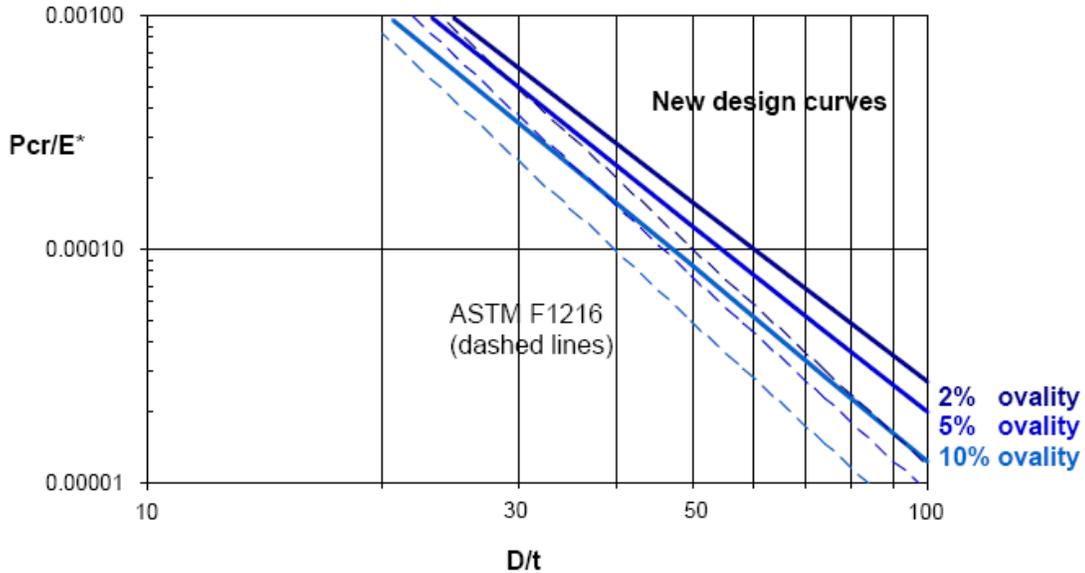


**Longitudinal** (e.g. flat invert due to residual sediment)

**a) CHARACTERISTIC**

**b) SYSTEM**

**Figure 8** – Examples of characteristic (renovation technique) and system (host pipe) imperfections affecting liner buckling resistance

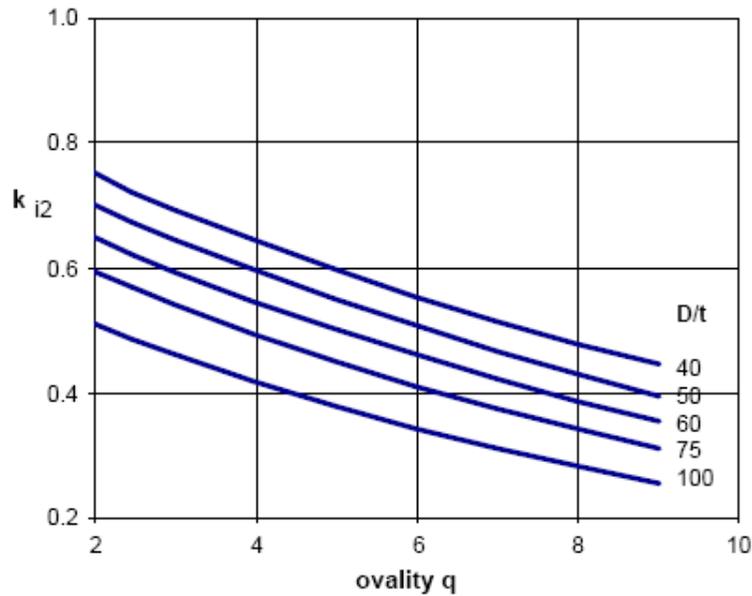


**Figure 9** – Typical form of new design chart for CIPP incorporating a characteristic gap imperfection (here 0.5%)

To appreciate the difference in outcomes using the new and old design curves in Figure 9, it should be noted that for CIPP the usual procedure is to determine the wall thickness  $t$  required to safely resist a given value of groundwater pressure. Since pressure is entered on the y-axis, and thickness read via the value of  $D/t$  from the x-axis, it is the horizontal rather than vertical separation of the two sets of curves, which is of practical significance. A simple example serves to illustrate the point.

For a 1200mm diameter sewer with 5% ovality under 6 meters sustained head of groundwater, the thickness of liner with long-term modulus  $E^* = 1200$  MPa (174.ksi) required to provide a factor of safety  $N = 2$  against buckling is determined as follows. The groundwater pressure  $P = 0.06$  MPa (0.008 ksi), so  $P_{cr} = N.P = 0.12$  MPa (0.017 ksi), and  $P_{cr}/E^* = 0.0001$ . Entering this value on the y-axis, the intercept with the new design curve for  $q = 5\%$  yields  $D/t = 60$ , while the corresponding ASTM F1216 curve gives  $D/t = 50$ . Since  $D$  is defined as mean diameter of the liner (i.e. measured to the centroidal axis of the wall), the corresponding values of standard dimension ratio  $SDR$  (= outside diameter/thickness) are 61 and 51 respectively. The required thickness by the new method is then  $1200/61 = 19.7$  mm (0.77 in), compared with  $1200/51 = 23.5$  mm (.925 in) by the old, a saving of 16%. For more lightly loaded liners, and/or stiffer materials, resulting in lower values of  $P_{cr}/E^*$ , the potential savings increase as the new and old design curves increasingly diverge.

Recognizing that accurate reading of log-log plots is not always as easy as in the above example, it is proposed in due course to derive simple curve-fitting formulae to enable programming of such a chart into a spreadsheet. The graphical presentation nevertheless provides a helpful overview of liner performance and the non-dimensional grouping of all plotted parameters allows any consistent set of units for each group to be used.



**Figure 10** – Combined imperfection factor on 2-lobe buckling pressure of perfect circular liner as a function of system ovality, for characteristic gap imperfection of 0.5%

Further specialization of the chart, i.e. beyond incorporation of a characteristic gap imperfection, would however be straightforward if desired. For example, by also building in a safety factor and the relevant characteristic modulus  $E^*$  of the product in question, the y-axis could be adapted to allow direct entry of the design water head in meters or feet, and the x-axis to read directly in *SDR* rather than  $D/t$ .

## 18.2 Close-fit thermoplastic pipes

For deformed/reformed liner pipes, charts similar to Figure 8 could be produced, incorporating characteristic longitudinal and gap imperfections as appropriate to each material and process. If, as is usual, the product is extruded and supplied in a discrete range of *SDRs*, an alternative graphical presentation is suggested. This would be based on a form of Eq. 3 written as:

$$P_{cr}/E^* = k_i \cdot c_0 (D/t)^{-2.2} \quad (\text{Eq. 4})$$

where  $c_0$  is the value of coefficient  $c$  for zero imperfection, and  $k_i$  is the overall reduction factor on buckling pressure. Eq. 4 is similar to the formula based on the original Glock analysis already advocated elsewhere (El-Sawy & Moore, 1997; ATV, 2000), with the important difference that  $k_i$  here is a single factor incorporating the combined effects of all imperfections. The fact, already demonstrated, that the power  $m$  of  $D/t$  in Eq. 3 is not constant but is itself a function of imperfections means that  $k_i$  also varies with some non-zero power of  $D/t$ .

Figure 10, developed using the same numerical data used to construct Figure 9, illustrates the principle of how Eq. 4 would be applied in practice. The factor  $k_{i2}$  plotted refers to 2-lobe behavior and is intended for use in Eq. 4 in conjunction with a value of  $c_0 = c_{02} = 1.323$ . The need to interpolate between the plotted curves for the required output parameter  $D/t$  makes this form of chart relatively impractical for design of CIPP, but if reproduced for thermoplastic pipes at relevant fixed values of  $SDR$  it would be quite suitable. For some close-fit pipe materials and processes, for example heat-reverted polyethylene, the  $SDR$  range would be lower, and typical gap imperfection due to thermal shrinkage higher at around 2% (Alberding & Falter, 1997), so the detail of the design chart would look quite different from Figure 10, although qualitatively similar. Any material-related longitudinal imperfection of folded pipe (Figure 8a) would in practice be indistinguishable from that of an additional characteristic gap, and could be incorporated accordingly. If measurements of pre-buckling deformations established that the overall characteristic imperfection made the one-lobe rather than two-lobe mode critical, design should also for consistency be based on the one-lobe factor  $k_{i1}$  (which in theory differs slightly from  $k_{i2}$ ) applied to Eq. 4 with  $c_0 = c_{01} \approx 1$ .

### 18.3 *Profiled and structured wall liner pipes*

A major advantage of the new theory is that it takes explicit account of the contribution of axial (hoop compression) as well as flexural strains on the stability of the liner pipe ring. For liner pipes of profiled or non-homogeneous wall construction the generalized form of Eq. 3 is:

$$P_{cr} / E^* = \lambda \cdot c \cdot \left[ D \sqrt{\frac{A}{12I}} \right]^m \quad (\text{Eq. 5})$$

$$\lambda = \sqrt{\frac{A^3}{12I}} \quad (\text{Eq. 6})$$

where  $A$  and  $I$  are respectively the area and second moment of area of the wall cross-section, and the “median” pipe diameter  $D$  is now measured at the neutral axis rather than at mid-thickness.

Thus a chart of the form of Figure 9 may readily be adapted for profiled wall pipes simply by re-labeling the axes as follows:

y-axis:  $P_{cr}/E^*$  replaced by  $P_{cr}/\lambda E^*$

x-axis:  $D/t$  replaced by  $D \sqrt{\frac{A}{12I}}$

Note that the simple substitution of thickness  $t$  by  $\sqrt{12I/A}$ , as referenced quite widely in the literature on linear buckling theory, is NOT sufficient. If this is done with the non-linear theory, as suggested by the new German ATV guide, it leads to the completely erroneous result that a profiled wall liner pipe has a higher buckling pressure than a solid wall pipe of the same flexural ring stiffness  $EI/D^3$ . In fact the complete opposite is true. This can be appreciated by the fact that a profiled wall pipe has relatively low cross-sectional area and hence axial compression stiffness, and that for perfectly close-fitting liners it is the compression stiffness  $EA$  rather than the flexural stiffness  $EI$  which dominates buckling response.

Correct interpretation of either the Glock (1977) or Boot (1998) analysis yields the additional factor  $\lambda$  applicable to the buckling pressure. Furthermore, the following expression can be derived for the buckling pressure of a profiled wall pipe relative to that of a solid wall pipe of the same flexural ring stiffness:

$$\frac{(P_{cr})_{\text{profiled wall}}}{(P_{cr})_{\text{solid wall}}} = \lambda^{[1 - (m/3)]} \quad (\text{Eq.7})$$

where  $\lambda$  is given by Eq. 6.

At zero gap,  $m = 2.2$ , and for the range of  $I$  and  $A$  values of standard wall profiles listed in ASTM 1741(1996), Eq. 7 yields factors on buckling pressure in the range 0.65 – 0.85. With increasing gap, these factors increase as  $m$  increases, eventually approaching 1 as the unrestrained condition, for which  $m = 3$  and the buckling mode is purely linear and flexural (see Figure 3a), is reached.

The same considerations apply to structured wall pipes for which the equivalent values of  $A$  and  $I/A$  assuming a uniform  $E$  value for all elements of the structure may be derived by the method of transformed sections as used for reinforced concrete design. For both profiled and structured-wall pipes, however, it is important to note that creep buckling may not be the critical limit state for resistance to external hydrostatic pressure. Depending on the detailed wall section design, failure may occur in the form of local rib buckling in profiled wall liners, or by delamination or shear in structured wall pipes, before the theoretical buckling pressure is reached. By contrast, for homogeneous, solid-wall, circular liner pipes of flexible plastics, material failure is invariably less critical than buckling (Boot, 1998; Boot & Gumbel, 1998).

## 19.0 Implications for characterization of renovation products

Key input parameters for the new theory, as for the old, are the values of short and long-term liner modulus. The fact that non-linear creep buckling primarily involves compression of the liner pipe wall, with flexure assuming dominance only when the total imperfection becomes large, has sparked much debate about whether the conventional 3-point flexural or parallel-plate ring stiffness tests are appropriate. Research on CIPP in this area continues (Boot et al., 2001), but hitherto the indications are that some form of flexural test yields lower bound, but not overly conservative,  $E$  values for design (Boot &

Javadi, 1998b). Some form of long-term testing is always necessary. Even where design for a short-term storm event, of the order say of 1-3 days, is critical, short-term laboratory tests of typically 1-3 minutes duration may not provide a creep modulus on the relevant time scale. The difference for some materials, for example polyethylene, can be very significant (Alberding & Falter, 1997).

Just as important as the material stiffness is the need to characterize the imperfections of each individual renovation technique and material. This is new. Initial annular gaps can be estimated from the material thermal expansion properties, but to the extent that they are process dependent should also be backed up by measurements. In all cases, but especially where non-measurable imperfection effects such as residual stresses may be involved, there is no substitute for external pressure testing of lined pipe samples to fully characterize both material and geometry. In this regard significant advance has been made since the early days of testing using perfectly circular casings which produced a large amount of scatter. By testing in casings with 5% ovality (Boot et al., 2001), the scatter band can be appreciably reduced. This is apparent already from the data plotted in Figure 4, and has been confirmed by more extensive recent tests (Seeman et al., 2001).

## **20.0 Next steps in method development and implementation**

As demonstrated in the first half of this part, the proposed new design theory is both more consistent and better substantiated by available experimental evidence than existing formulae for restrained hydrostatic buckling of circular liners. This alone is in principle sufficient to justify immediate implementation of the new approach. Some further work is however needed to develop in detail the various design charts and other tools outlined in the second half of the part for practical application across the full range of renovation techniques discussed.

The choice of how best to present the methodology in each case will depend on the extent of available measurement and test data, as well as regional preferences and practice. In considering a suitable overall approach for application in North America, it is suggested that the ASCE PINS committee should proceed, in close collaboration with associated international research and standardization initiatives, as follows:

- a) Decide on suitable groupings of renovation techniques for which common design tools can be developed. Ideally the classification of renovation technique families in the process of being adopted as a European standard (CEN, 2001) should be followed.
- b) Establish minimum testing requirements for each technique family and liner material, and/or assign appropriately conservative default values to all relevant design characteristics, including characteristic imperfections, for which test data are not yet available.
- c) Publish fully detailed design procedures for beta-testing by both user and producer sides of the industry, which should also provide a consistent and balanced framework for validation testing of all individual products covered.

## 21.0 Concluding Remarks

1. Although it has served a useful purpose in helping to launch the renovation industry, the original WRc/ASTM hydrostatic buckling formula has been shown to be too conservative for current needs and above all limited in predictive power by its lack of theoretical consistency.
2. The adoption of an extensively researched and fully consistent design theory, already well substantiated at least for cured-in-place pipe liners, is now advocated. The principles of the theory have been explained, and its adaptability to all other close-fitting and grouted lining systems, including those using profiled or structured wall pipes, has been demonstrated.
3. With the aid of the new theory, the influence of gap, ovality and longitudinal imperfections on restrained hydrostatic buckling pressure has been described, both qualitatively and quantitatively. A new classification of imperfections as either characteristic to a particular renovation technique or arising from the host pipe system has been proposed.
4. Practical forms of design chart have been presented which already incorporate the characteristic imperfections of an individual renovation technique and relate the critical short or long-term water head to the liner wall thickness or ring stiffness according to the dimensions and system imperfections of the pipe to be lined. The derivation of alternative, but similarly transparent, spreadsheet design tools has been discussed.
5. The implications of the new theory for the characterization of renovation products have been briefly reviewed. Related improvements in testing technique are being developed as part of ongoing research projects at universities in the UK and USA.
6. The next steps to be taken by the ASCE Pipeline Infrastructure (PINS) committee in considering how the proposed new methodology might be implemented in North America have been outlined.

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