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Rehabilitation of Fully Deteriorated Rigid Pipes by Flexible and Rigid Liners

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Abstract

This paper addresses appropriate analytical models for both flexible and rigid liner designs for the fully deteriorated pipe condition. The results of tests conducted in the Buried Structures Laboratory at Utah State University on one flexible (CIPP) and one rigid (GIP) liner in cracked concrete pipes are reviewed and are shown to support the analytical models developed. Also, two little known (in the USA) reports on tests conducted in the UK in the early 1980's are reviewed and used to illuminate the technical issues under discussion. Unresolved issues are highlighted and the need for well-conceived and controlled experimental research is discussed.

Introduction

For the past three years 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 fiber glass pressure pipe (AWWA C 950). The modification introduces an ovality factor that is not included in AWWA C 950. 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 a "worst case" design. Often the interpretation of this definition is that the designer must assume the host pipe has zero structural capacity. This paper discusses some logical inconsistencies in this design method and raises questions regarding the "worst case" assumption.

Schrock and Gumbel (1997) argue convincingly that the soil surrounding the deteriorated host pipe being rehabilitated 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 paper will not discuss this topic further. Because this paper 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 3 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 inch crack and its D-load to produce the ultimate load. The ultimate load indicates failure of the steel reinforcement while the 0.01-inch 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-inch 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-inch 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.

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 obtain 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.

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 (theoretically) separate pieces. This is the geometrical configuration of the liner 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 then justifies using the modified Luscher buckling equation of F

1216 (and AWWA C 950). 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.

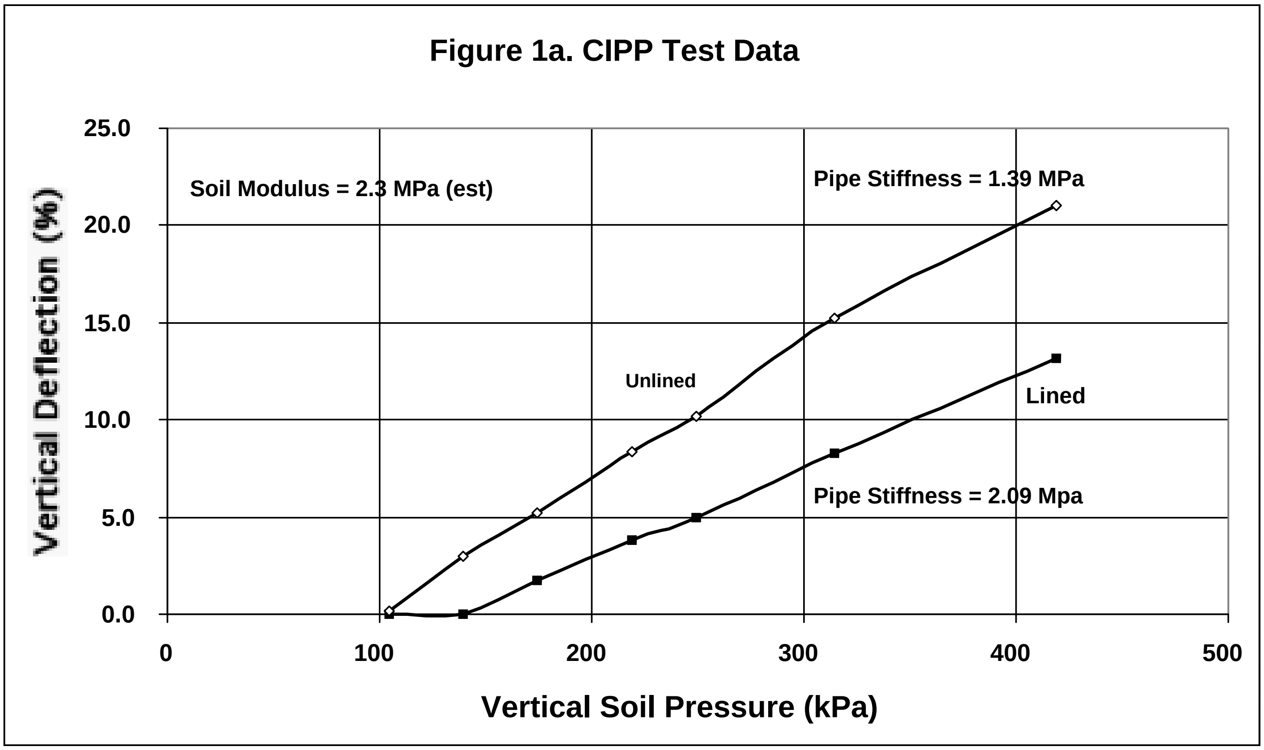
Utah State University Test Results

Figure 1.0 shows the results of testing at Utah State University on 750-mm cracked non-reinforced concrete pipe, both lined and unlined, in their large soil cell. Figure 1a. gives the results for a 21-mm CIPP liner (Watkins 1988) whose short-term modulus of elasticity was assumed to be 2,759 MPa. The cracked pipes were installed and the soil cell pre loaded to 69.6 kPa 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 Es' of from 690 kPa to 2.76 MPa. 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 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 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 to 2.76 MPa 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 2,759 MPa while that of the grout was estimated at 25,690 MPa. The moment of inertia equivalent thickness of the profiled PVC liner was 9 mm.

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.



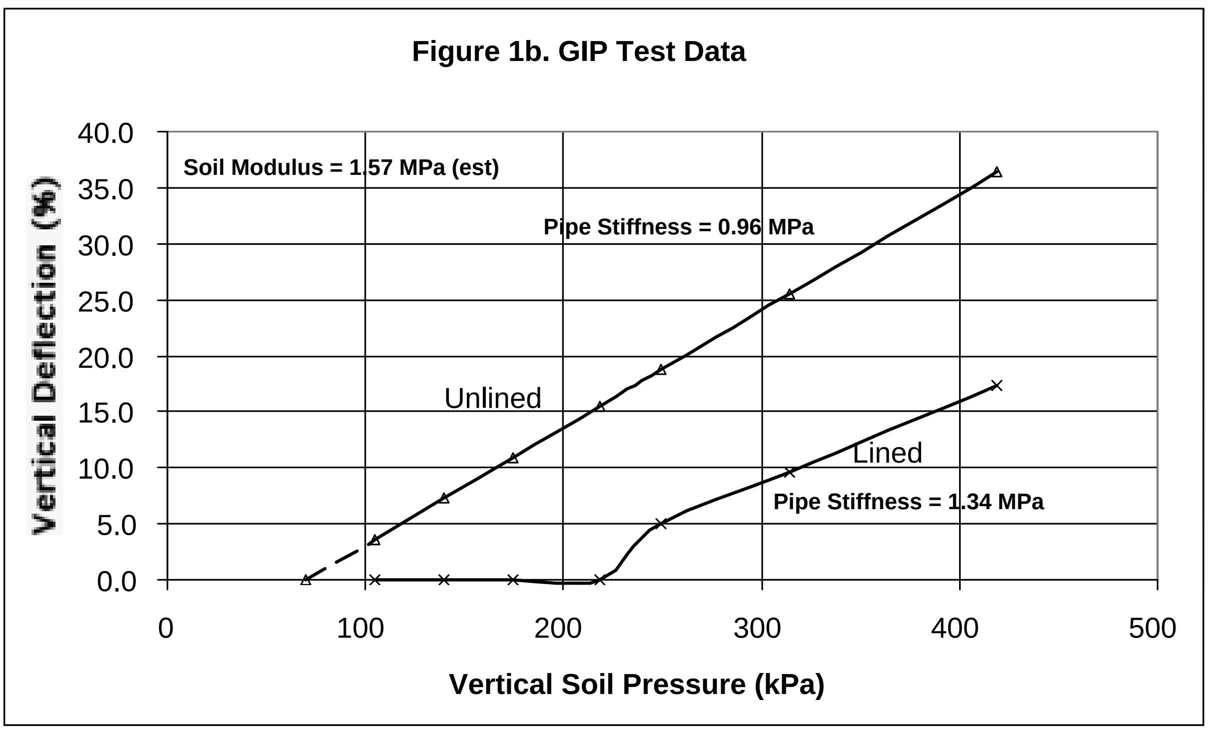


Figure 1. Load – % Deflection Curves from Utah State University Test

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 for the CIPP test and 1.6 MPa 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 x 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.

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 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 to over 30-mm) to produce the same 50% increase in combined pipe stiffness as obtained in soil of 2.3 MPa modulus. Another 8-mm 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! This liner would have a pipe stiffness of about 1.2 MPa; it would deflect 5% under 61-m of 1.5 kg/m^3 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 (Es' = 6.9 MPa) 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 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).

Water Research Centre (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) with CIPP, and GRP (grouted) liners and cracked concrete (875-mm) pipes with GRC (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. Under the same load pressure in "Poor" soil, the 14-mm 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 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 grout. The GRC liner bonded to higher strength grout (12 MPa) had a deflection of 0.08% while the deflection of the GRC liner with the lower strength grout (8 MPa) was 0.16%. The unlined cracked concrete pipe deflected 0.56% under the same vertical pressure of 100 kPa 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 C 950) 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 (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 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 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 CIPP with grouted annulus, 2) repointed brickwork, 3) original brick sewer, 4) 12-mm 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 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 to 11 MPa. 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. 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 ungrouted 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) 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). 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 ungrouted 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.)

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 2 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 – 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.

Conclusions

This paper 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 C 950 soil buckling. In fact, the tests in the abandoned egg shaped brick sewer lined with an ungrouted 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 can 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, some good academic research is needed on liner-cracked rigid pipe-soil interactions, deformations and stress resultants.

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