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“Fully Deteriorated” Design in Rigid Pipe Rehabilitation with Flexible Liners

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**Abstract**

An ASCE/PLD 2001 Conference paper by the author (McAlpine 2001) discussed available test data, research findings and technical discussions by various groups on the subject of “Fully Deteriorated Design” for rigid pipe rehabilitation by flexible liners. The paper concluded that there is no experimental or theoretical support for the current design method as it would apply to flexible liners of buried rigid pipes. The design equation, appropriated from the design practice for direct burial of fiberglass pipe, is based on a phenomenological model of the soil- pipe-liner structure that is inappropriate for rehabilitation liners in rigid pipes.

This paper further argues that, under most realistic field conditions, there will be no earth load transferred to the liner. In addition, in the most unusual and unlikely cases that could produce load transfer (non existent or unstable soil support), the load will produce pure bending stresses and buckling will not be the failure state (as assumed in the current practice). A deflection control design method is proposed for “weak” soils and host pipes that have lost their rigidity.

**Introduction**

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 and AWWA M45). The modification introduces (likely improperly) 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

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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 shows that the most likely scenario produces no earth load transferred to the flexible liner and certainly no thrust loads that could cause geometric instability (buckling).

The 2001 paper (McAlpine 2001) concluded that there is no experimental or theoretical support for the current design method as it would apply to flexible liners of buried rigid pipes. The design equation, appropriated from the design practice for direct burial of fiberglass pipe, is based on a phenomenological model that is inappropriate for rehabilitation liners in rigid pipes.

### **Material Characteristics**

The earlier paper also discusses the well-known (Serpente 1994) failure modes (flexural cracking) of rigid pipes such as vitrified clay (VCP), nonreinforced (ASTM C14) and reinforced (ASTM C76) concrete pipes. By their nature these rigid pipes have high compressive strength but are relatively weak in flexure. Further, they have high elastic moduli (3-5 million psi) that results in low strain tolerance. For example, the tensile/flexural strength or “modulus of rupture” can be calculated as  $f_r = 7.5 \sqrt{f_c'}$ , where  $f_c'$  = 28-day compressive strength in psi. Thus a concrete structure with  $f_c' = 4,000$  psi has a “modulus of rupture” of 474 psi. (NOT zero as often assumed to simplify complex calculations!) The elastic modulus can be estimated as  $E_c = 33(145)^{1.5} \sqrt{f_c'}$  where 145 is the density of the concrete in pounds/cubic foot (PCF). Thus, for  $f_c' = 4,000$ ,  $E_c = 3,644$  ksi. Using these characteristics of concrete we can estimate the maximum strain (at failure) in both flexure and compression as 130 and 1100 microstrains (in/in  $\times 10^{-6}$ ), respectively.

In contrast to concrete, plastics, such as flexible pipe liners, have relatively low elastic moduli (about 10% of concrete), high flexural strength (compared to concrete and VCP) and are very strain tolerant. Not surprisingly, these are two very different engineering materials. The relevant question for this paper is “When put together in a single structure, how do they interact when loaded?” If the flexible liner is bonded to the concrete, either mechanically or chemically so that the strains in the two materials are equal at the interface, then the structure can be considered a “composite material structure” and analyzed by the method of “transformed-sections” (Ugural & Fenster 1987, pp. 156-162). The analytical procedure is to “transform” one material into an equivalent (in an engineering mechanics sense) amount of the other material and the resulting structure is analyzed as a homogeneous material. For example, a 1.0 inch thick flexible liner bonded to the inner surface of a concrete pipe would be analyzed as a concrete structure 1.0  $(E_p/E_c) = 0.1$ -in thicker than the concrete alone. The transformed-section method also computes the position of the neutral axis (line of zero strain) and the moment of inertia of the composite structure. A series of tedious but straightforward calculations for the 1.0-inch liner bonded to a 2.0-inch concrete pipe wall yield a new neutral axis about 7% closer to the liner-pipe wall interface and a composite moment of inertia about 33% greater than the concrete wall without the liner. Thus, flexural stress  $\sigma = My/I$  is reduced by about 25% for any additional

moment applied after the liner is installed. It should be noted that this example calculation corresponds, roughly, to a liner DR = 24, at the lower range of most liners with  $E_p = 0.1 E_c$ . For liner DR = 48, the increase in moment of inertia is less than 12% and the reduction in future flexural stress is about 10%. Due to strain incompatibility, it is highly unlikely that concrete compressive strains induced after lining would be large enough to cause liner buckling. In fact, the likelihood of any change in pipe wall stains after lining is very small and would have to be caused by changes in the load or soil support.

For the pipe rehabilitation situation, it is reasonable to assume some loss of pipe wall thickness due to corrosion of the concrete. Thus, assuming loss of 1.0-inch of concrete pipe wall and using the relation from ASTM C76 that pipe wall thickness (Wall B)  $T = \text{Diameter}/12 + 1.0$ , estimating the remaining wall thickness  $T' = D/12$ , where  $D =$  new inside pipe diameter in inches is a reasonable estimate. Combining this with the relation  $DR = D/t$  where  $t =$  thickness of liner in inches, we have  $T'/t = DR/12$ . Thus for a given DR, the ratio of concrete wall to liner thickness is roughly the same for all diameters and the effect on the composite characteristics should be the same in terms of percentage change in neutral axis and moment of inertia. Obviously, the percent increase in moment of inertia decreases as the DR increases (at DR = 80 the increase is less than 6%).

### **Soil Loads on Rigid Pipes**

The vertical soil load capacity of a rigid pipe is dependent on the soil support in the immediate vicinity of the pipe, i.e. bedding and haunch support. The crown bending moment magnitude, and thus the flexural stress, can vary by as much as 4-to-1 depending on the quality of soil support. Clearly, any deterioration in this soil support can lead to flexural failure of the pipe. This soil support deterioration can be caused by ground water infiltration carrying soil into the pipe through cracks in the pipe and/or loose joints. This points out the primary importance of the rehabilitation liner effectively sealing the pipe and preventing ground water infiltration. Further, it highlights the potential importance of determining the state of the soil support before designing the rehabilitation liner. This is hardly ever done because it is technically difficult and expensive. In addition, the current design method for “fully deteriorated” pipes may not have emphasized enough the soil support required for the method to be applicable.

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. It should be recognized that this argument may not be valid in certain soils, e.g., cohesionless soil. The Schrock-Gumbel

argument stated above must surely assume the stability of the pipe-soil structure, i.e., the pipe-soil system is in equilibrium.

### **Pipe Structure Losses**

Obviously, the pipe can also lose soil load capacity by losing structural strength; for example by the loss of pipe wall thickness by corrosion due to H<sub>2</sub>S gas. For nonreinforced rigid pipe the flexural stress  $\sigma = 6M/T^2$  and a 25% loss of wall thickness T would increase the stress  $\sigma$  by 78%, assuming the bending moment M remains constant. A 30% loss would more than double the stress level. Generally, corrosion is not a problem with VCP but is for concrete pipes. When a nonreinforced rigid pipe cracks (flexural failure) it loses its ring stiffness and its deflection is entirely dependent on its soil support. In concrete pipe terms, its D-load strength is equal to its ultimate strength (in three-edge-bearing, TEB, tests). As clearly demonstrated in the Utah State University soil cell tests (Insituform 1988, Danby 1993, see McAlpine 2001), buried cracked nonreinforced concrete pipe act as a flexible buried pipe with a linear increase in % deflection with increased vertical loads. The slope of the load-deflection line (the stiffness of the pipe-soil structure) is dependent on the soil stiffness (modulus of soil reaction  $E_s$ ). The maximum deflection is limited by the collapse of the pipe when the pipe segments lose physical contact with each other, especially in the crown. This occurs when the vertical deflection of the crown segments equals the pipe wall thickness, 8 – 12% of diameter, depending on wall type or pipe class. The unlined cracked test pipes used in the Utah State University Soil Cell Tests (McAlpine 2001) deflected only about 5% (10% loss of vertical diameter) with an increase in vertical load of 200—250%. This represents a significant residual structural capacity as a flexible structure.

Reinforced concrete pipe (RCP) generally (ASTM C76) has an ultimate strength about 50% greater than its D-load strength (point at which flexural cracking occurs, producing the famous (and arbitrary) “.01-inch crack”). It is important to note that flexural cracking in RCP is not considered as pipe failure. Indeed, the classical “indirect design” of RCP allows a vertical load (adjusted for bedding factor) equal to the D-load value. This design method, in effect, relies on the additional load capacity provided by the reinforcement as its safety factor (SF = 1.5). Watkins and Anderson (2000, p.70), in their excellent and unique textbook, give an example calculation that shows that at incipient flexural cracking in a 36-inch RCP under three different loading conditions the deflections are 0.06%, 0.05%, and 0.10%. From the data given, one can calculate the stress in the reinforcement to be 3,600 psi, less than 10% of the assumed 42-ksi tensile strength of the steel.

### **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 or some combination of these events, after the rehabilitation liner installation.

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. Current design practice assumes that soil support is constant (i.e., is the same after as before rehabilitation) as well as producing only uniform radial pressure on the liner. Note that, if this assumption were valid for the rigid host pipe, the pipe would not fail because all stresses would be compressive. If the rigid host pipe has not failed catastrophically (become “fully deteriorated”) before liner installation, there is no logical reason to assume that it will reach that state of deterioration after liner installation. If there are sound engineering reasons to expect the pipe-soil structure to collapse at some future time, it is unlikely that any flexible liner will mitigate the geotechnical problems. The concept of “fully deteriorated” design should be abandoned.

### **Design Cases**

Recall that we are concerned in this paper with the class of rehabilitation systems generally described as “close fit, flexible liners” such as CIPP and Fold and Form. Further, we will make the reasonable assumption that these liners do not bond reliably to the walls of the rigid host pipe and, therefore, do not form a composite structure. As shown earlier, because of the large difference in modulus values between the plastic liners and rigid pipe materials, even bonded liners produce only marginal structural improvements. Also, due to the fact that most of these liners shrink a small amount after installation, it is highly unlikely that any initial bonding will survive

these shrinkage stresses. Thus, it can be assumed that no strains will be produced in the liner due to strains in the host pipe wall after lining is completed. Further, any loading on the liner must result from deflection of the host pipe after liner installation. This type load on the liner would produce deformation of the liner geometry but would not produce force loads. This is because the deflection of the host pipe would, in the vast majority of cases, be limited by the pipe-soil structure reaching a new equilibrium condition essentially independent of the presence of the liner. Certainly this would be true of stiff soils, say  $E_s' > 400$  psi.

In order to quantify a range of design cases for evaluating proposed design methods, we choose to classify by crown crack width “w” of the rigid host pipe prior to lining. We know that cracking of concrete pipe (at its D-load strength load in TEB test) occurs when  $w = 0.01$ -inches. By use of simple proportionality we can deduce that  $w = 0.17$ -inch when 65 ksi strength rebar steel yields. Further, we know that failed rigid pipe (nonreinforced or reinforced) stops acting as flexible pipe when its segments loose physical contact,  $\Delta = T$ , where  $\Delta$  is the vertical deflection of the pipe crown at the crack and T is the pipe wall thickness. Thus, we will define three pipe conditions: 1)  $0 \leq w \leq 0.01$ , 2)  $0.01 \leq w \leq 0.17$ , and 3)  $0.17 \leq w$  and  $\Delta \leq T$ . In addition we will classify soils as “weak” ( $E_s' < 400$  psi) and “stiff” ( $E_s' \geq 400$  psi).

Type of Pipe	$0 \leq w \leq 0.01$	$0.01 \leq w \leq 0.17$	$0.17 \leq w \leq 0.17$	$0.17 \leq w$ & $\Delta \leq T$	$0.17 \leq w$ & $\Delta \leq T$
		<b>Weak Soil</b>	<b>Stiff Soil</b>	<b>Weak Soil</b>	<b>Stiff Soil</b>
<b>Clay/NRCP</b>	H2O Only	Limit Deflection	H2O Only	Limit Deflection	H2O Only*
<b>RCP</b>	H2O Only	H2O Only	H2O Only	Limit Deflection	H2O Only*

\*Include deformation of liner geometry

**Table 1. Proposed Design Method Matrix**

Some research is required to determine the best design formula for limiting deflection of the liner-pipe-soil structure. The Utah State University Soil Cell tests (McAlpine 2001, Watkins and Shupe 1988, Watkins 1993) showed some correlation to use of the Iowa Deflection Formula but these tests were very limited and performed only in “weak” soils. Clearly, the design objective would be to prevent the deflection causing physical separation of the cracked pipe segments, i.e., insuring total  $\Delta \leq T$ . However, the designer must also be given some assumption about loading that the liner would be resisting. Because we have assumed that the pipe-soil structure is stable (in equilibrium), any assumed force loads on the liner are purely hypothetical. Perhaps a way around this conundrum is to assume that the current pipe-soil structure has experienced a vertical deflection at the crown of  $\Delta_c$  and the design objective is to limit future deflection to  $T - \Delta_c$  due to an increase in load of, say, 200%.

If we postulate that use of the Iowa Deflection Formula is acceptable in this application (Watkins and Anderson 2000, Appendix B), it can be shown that  $(T - \Delta_c)/\Delta_c = 2 / (R_s + 1)$ , where  $R_s = (E \cdot I / r^3) / .061 E_s'$ , i.e., the ratio of the pipe

stiffness to the soil stiffness terms in the Iowa formula. Figure 1 shows the pipe stiffness (PS) required of the liner for various values of  $T/\Delta c$ . Figure 2 shows the same information in terms of liner SDR required assuming a liner modulus  $E = 150,000$  psi. Note that the PS required goes to zero when  $\Delta c = T/3$  because the pipe-soil structure stiffness is adequate to limit additional deflection to  $2T/3$  for a increase in load of 200% (i.e., no liner required for deflection control under the stated design objective). As  $T/\Delta c$  approaches 1.0 the liner stiffness required increases rapidly because there is little additional deflection allowed and the pipe-soil stiffness has already allowed a large  $\Delta c$  (thus  $R_s$  must be very large).

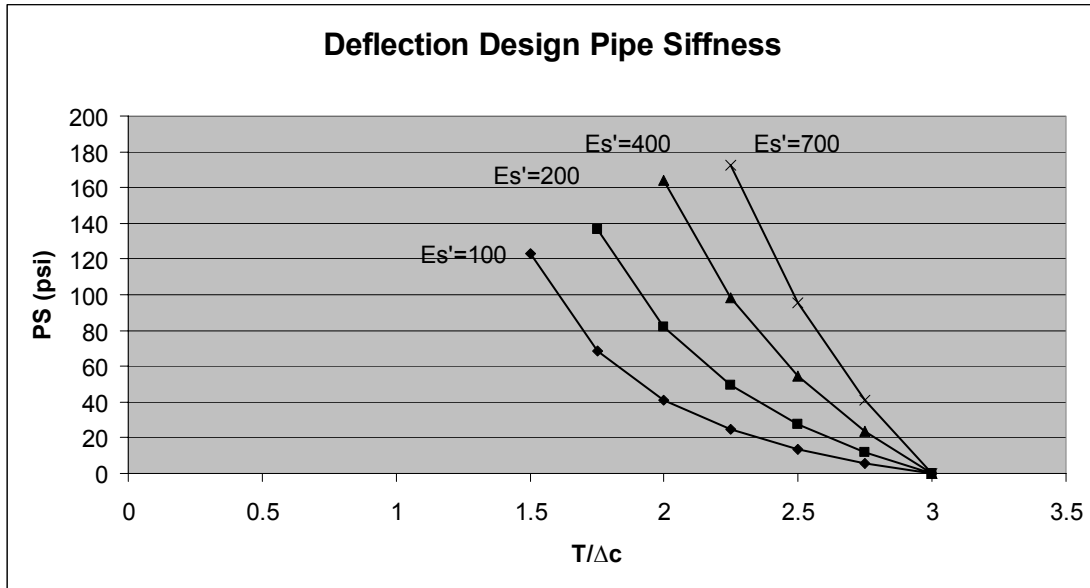


Figure 1. Liner PS Required to Limit  $\Delta \leq T$

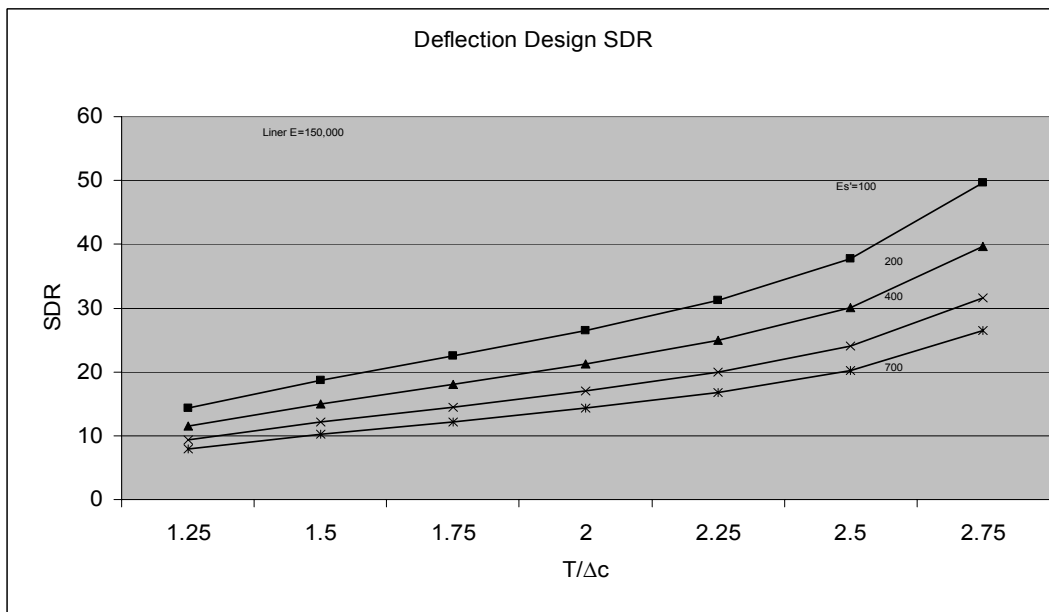


Figure 2. Liner SDR Required to Limit  $\Delta \leq T$

It should be noted that the value of  $\Delta_c$  at which required  $PS = 0$  depends on the assumed increase in load for which  $\Delta = T$ . For example, if the increase in load is chosen as 100% then the critical current deflection  $\Delta_c = T/2$  (instead of  $\Delta_c = T/3$ ). Also, the design criteria that allows for maximum deflection  $\Delta_{max} = T$  is arbitrary and could be changed to any smaller value deemed appropriate. These choices of design criteria are appropriate subjects for consensus discussions of industry designers such as ASTM F 17.67. The determination of the most appropriate design equations requires further research involving soil cell testing guided by detailed computer models and Finite Element Analyses. Further these research studies would validate or invalidate the approach to design proposed in this paper.

## Conclusions

The Utah State University soil cell tests showed that buried cracked nonreinforced concrete pipe acted as a flexible pipe with significant load carrying capacity. The 1993 test (Watkins 1993) showed that the unlined cracked pipe (actually, pipe-soil structure) had a pipe stiffness (PS) of about 140 psi in “weak” soil ( $E_s'$  estimated at 228 psi by use of the Iowa Deflection Formula). The deflection was approximately linear as a function of vertical load over a load range of over 3:1. The 1988 tests (Watkins and Shupe 1988) was performed with the soil in the cell precompacted some (to 2% pipe deflection) before data were taken. This precompaction increased the estimated  $E_s'$  to about 331 psi and the measured unlined pipe stiffness to 202 psi. Both tests showed that flexible liners can increase the effective pipe stiffness in these soils (50% and 40%) and that there was reasonable correlation with the Iowa Deflection Formula. Both tests showed the segments of broken concrete pipes transfer the soil pressure to the liners as line loads (bending not thrust loads). In fact, this was confirmed in the 1993 test that employed strain gages on the liner.

However, the most important conclusion from the Utah State University tests is that pipes that would certainly be classified as “fully deteriorated” under current design practice, have significant residual load carrying capacity even in “weak” soils. Ignoring this residual structural capacity is unrealistic and the practice should be discontinued. Use of liners in such pipes (with adequate residual structural capacity) can be justified on the basis of preventing the deterioration of soil support by ground water infiltration and designed for adequate hydrostatic buckling strength (Gumbel 2001). Further, the design for deflection control proposed here is based on measuring  $\Delta_c$ , the current vertical deflection at the crown and/or the crack width  $w$ , that are directly measurable in man-entry pipes and readily estimated in smaller pipes. The other basic assumption made here is that the pipe-soil structure is in equilibrium or, if not, will reestablish such equilibrium with only small deflections. If this assumption is not valid, then deflection control design would have to be based on liner PS alone using loading models approaching that of TEB tests because of the manner in which the remnants of the host rigid pipe transfer earth loads. In the vast majority of real design cases this would be unreasonably conservative. Also, it should be recognized that current design practice (ASTM F1216) implicitly assumes pipe-soil stability (equilibrium) as well as two additional conditions that are clearly impossible. First, it



assumes the rigid host pipe is replaced by native soil in intimate contact with the liner and second, the soil pressure on the liner is uniform and entirely radial.

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