DESIGN OF LINERS FOR DETERIORATED SEWERS – LATEST RESEARCH TO MAKE IT MORE EFFICIENT

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ABSTRACT

Since the start of the Trenchless Technology industry, liners for deteriorated sewers have been structurally designed by applying Standards and methods that apply to flexible pipes installed by traditional trenching. It has always been known that there are large gaps in the logic of doing this, but in the absence of consensus on any alternative design method it has become the norm.

The load carrying capacity of a flexible pipe requires consideration of its strength and the support it receives from its surroundings. The situation is more complicated for a liner as the deteriorated condition of the host pipe means that the support for the liner cannot readily be determined.

This Paper details research to determine the load bearing capacity of grouted spiral liners installed in deteriorated gravity pipelines. The research aimed to quantify by how much grouting increases the load carrying capacity of a spiral wound liner compared to an ungrouted liner and also to an unsupported liner inside a rigid host pipe.

The results of testing have shown that the effect of grout is far greater than previously considered. The implication of this work is that the cost and feasibility of structural renewal of pipelines can be greatly reduced. The research is particularly relevant to earthquake zones, and also allows more efficient design of liners for larger diameter pipelines.

KEYWORDS

Sewer rehabilitation, relining, spiral wound liners, grouting, structural liner design

1 INTRODUCTION

Since the start of the Trenchless Technology industry, liners for deteriorated sewers have been structurally designed by applying Standards and methods that apply to flexible pipes installed by traditional trenching. It has always been known that there are gaps in the logic of doing this, but in the absence of consensus on any alternative design method it has become the norm.

Despite these anomalies liners are performing well. We seemingly never hear of properly installed liners failing when subjected to the loads for which they have been designed.

But are these design methods efficient? Are current Specifications resulting in overdesigned sewer liners that are unnecessarily expensive to provide?

Structural design to determine whether the proposed liner has sufficient strength to carry applied loads obviously requires consideration of the liner characteristics and the support it receives from its surroundings.

The situation is more complicated than for the design of buried flexible pipes as the deteriorated condition of the host pipe means that the support for the liner cannot readily be determined. Deteriorated sewers, typically rigid pipes, may be cracked, corroded, displaced or partially collapsed. They come in an infinite range of conditions and internal shapes. Support provided to the liner may not be uniform along its length and around its circumference, meaning a fundamental assumption applied in flexible pipe design is invalid.

Most research has focussed on cured-in-place liners that are installed in a soft condition, moulding to the shape of the host pipe as they are internally pressurised and cured. Liner design methods have been adapted using this research.

Spiral wound or wound-in-place liners provide a circular cross section and uniquely allow the possibility of encasement with cementitious grout after installation. Clearly this grout encasement strengthens and supports the liner, increasing its load carrying capacity. Assumptions are typically applied to account for this strength increase, but research to provide validation has not been carried out until now.

2 ISSUES WITH DESIGN METHODS

Two design methods are routinely applied in sewer lining Specifications in Australia and New Zealand, depending on the assessed condition of the host pipe.

2.1 FULLY DETERIORATED DESIGN

The "fully deteriorated" design method assumes that the host pipe has no remaining load carrying capacity and so all loads from soil, groundwater, vehicles etc must be taken by the liner.

The equations used to determine the liner's load carrying capacity are taken from either AS/NZS 2566.1:1998 Buried Flexible Pipes - Part 1: Structural Design or the ASTM Standard Specification for the type of liner proposed.

These two Standards use differently formatted equations, but basically agree that the load carrying capacity of a flexible pipe is a function of the combination of the pipe stiffness and the strength of the embedment surrounding it.

When applied to liner design, no consideration is made of the host pipe surrounding the liner. It is not only assumed that the host pipe has no remaining strength, but also that the host pipe has completely disappeared. It is assumed that the liner is surrounded and supported only by the embedment that originally surrounded the host pipe.

The design method calculates deflection and wall strain in the liner and determines the factor of safety against buckling under full soil and water loads.

There are very few situations where this scenario realistically applies because:

1. The host pipe, while deteriorated, has not collapsed, so is therefore withstanding the applied loads, although with a reduced factor of safety. Assuming the liner takes all applied loads is therefore obviously conservative.
2. Flexible pipe design theory concludes that the stiffness of the embedment around the pipe has the greatest influence on its load carrying capacity. This does not directly apply to liners as the host pipe effectively confines the liner, isolating it from the surrounding embedment.

2.2 PARTIALLY DETERIORATED OR INTACT DESIGN

The "partially deteriorated" or "Intact" design scenario considers that the host pipe confines the liner. The host pipe is considered capable of supporting loads from soil and vehicles, so that the load on the liner is only from groundwater infiltration from cracks or leaking joints.

Confining the liner prevents it from deflecting, but as the applied load leads to wall compression, it can fail by buckling.

The Partially Deteriorated design scenario adapts the Timoshenko equation for the buckling resistance of a cylinder subjected to uniform hydrostatic pressure.

This design method can be considered more realistic for liner design as it as it acknowledges the role of the host pipe in confining the liner and isolating it from the surrounding embedment.

Timoshenko's equation for the buckling capacity of a cylinder can be written as:

$$q = \frac{24 \times S}{\left(1-\upsilon^2\right)} \quad \ldots (1)$$

Where: q = applied load

S = Stiffness of the flexible pipe, determined by calculation or testing

υ= Poisson's ratio for the pipe material

For liner design Specifications the equation is modified with additional terms added:

$$q = \frac{24 \times C \times K \times S}{N \times \left(1-\upsilon^2\right)} \quad \ldots (2)$$

The additional terms in this equation that specifically apply to liner design are:

C = a factor to account for ovality of the liner, noting that a cylinder with an oval shaped cross-section has reduced buckling capacity compared to a circular cylinder.

K = a factor to account for the enhancement to the buckling capacity of the liner due to the the restraint provided by the rigid host pipe. The value of K is dependent on how tightly the liner is held by the host pipe. It is varied to account for any gap between the liner and the host pipe, noting that the smaller the gap, the more enhancement is provided against buckling.

N = Factor of safety. Typically a value of 2 is specified.

The ovality factor C is determined from the following equation given in the relevant ASTM Standard Specifications and Australian and New Zealand Water Authority Specifications:

$$C = \left(\frac{1-\Delta}{\left[1+\Delta\right]^2}\right)^3 \quad \ldots (3)$$

Where Δ = ovality of the liner = (Mean inside diameter – minimum inside diameter) / (Mean inside diameter)

Typically Australian and New Zealand Water Authority Specifications require the value of K to be equal to 4, or 7 when the liner is grouted.

The above design equation has been modified by research conducted mainly in Europe which, while including terms for ovality and degree of fit as above, also includes a load capacity reduction factor to account for a "wavy imperfection" which is considered to be an inherent feature of liners which are installed in a soft state. Testing has shown that even a small "wavy imperfection" can cause a major reduction in load carrying capacity. This factor is not considered in Australian and New Zealand Water Authority Specifications, even though this defect is prevalent in cured-in-place liners.

3 THE TEST PROGRAM

3.1 AIMS OF THE TEST PROGRAM

Testing has previously been carried out to determine the support provided to cured-in-place liners and the results have generally been applied to the design of all liners.

Until now there was little understanding of how restraint by the host pipe affects spiral wound liners. In particular no attempt had been made to quantify the increase in enhancement provided by grouting.

The testing carried out by Interflow aimed to compare the load carrying capacities of a spiral wound liner in 3 scenarios:

1. Unsupported – not in a host pipe
2. Liner in a host pipe, spirally wound to contact the host pipe wall – not grouted
3. The same liner in 2 (above) but grouted.

The load carrying capacities obtained would be compared with calculated values, with back calculation being used to determine realistic values of enhancement (K) which could be applied in the liner design equation (Equation (2), above).

3.2 TESTING PROCEDURES

3.2.1 THEORY BEHIND THE TESTING

Testing was carried out on a Rotaloc spiral wound liner made with 91-37 UPVC profile. Rotaloc liners have been installed by Interflow in deteriorated sewers with diameters from 1050mm to 1,800mm.

For the purposes of this testing the Rotaloc liner was wound at an external diameter of 1,200mm. The stiffness of a 91-37 Rotaloc liner wound at a diameter of 1,200mm has previously been tested in accordance with ISO 9969 and found to have a value of 1,559 N/m/m.

Poisson's Ratio (υ) of UPVC is given in Table 2.1 of AS/NZS2566.1 as 0.38.

These values can be substituted in Equation 1 (above) to calculate the expected load that would cause the liner to buckle, as follows:

$$q = \frac{C}{N} \times \frac{24 \times K \times 1{,}559}{\left(1 - .38^2\right)} \times 10^{-3} \text{ kN/m}^2 \qquad \text{... (4)}$$

For an unsupported liner, the appropriate value of K = 1. Applying a factor of safety N = 1, bucking can be expected to occur at the following loads when the liner has the ovalities shown in Table 1

Table 1: Load to cause full buckling, calculated in accordance with Equation (4) above.

% Ovality	Calculated "C"	Calculated Buckling Load
0%	C=1	q = 43.7 kN/m^2
2%	C = 0.84	q = 36.7 kN/m^2
5%	C = 0.64	q = 28.0 kN/m^2

3.2.2 TESTING THE UNSUPPORTED LINER

The "Unsupported test" proceeded as follows:

1. A cylindrical "pressure vessel" was fabricated with:
   Internal diameter = 1,500mm
   Length = 6,000mm
   Wall thickness = 12mm

2. A Rotaloc liner with an external diameter of 1,200mm was wound into the pressure vessel. The liner was centred in the pressure vessel, supported by 2 wooden rails running along its length.

3. Steel end rings were placed around the ends of the Rotaloc liner to allow assembling of the sealing bungs. (See Pictures below)

4. The sealing bungs were placed around the ends of the liner, bolted to the flanges on the pressure vessel. This provided a water-tight space between the outside of the liner and the inside of the pressure vessel.

5. Indicator bars were hung inside the liner in 3 positions and in 4 planes at each position. The bars were hung so that the liner would contact the end of one of these bars when its deflection reaches 3%. See Picture 3. This 3% deflection was taken as "buckling failure" to provide a consistent point for comparison in each of the test scenarios.

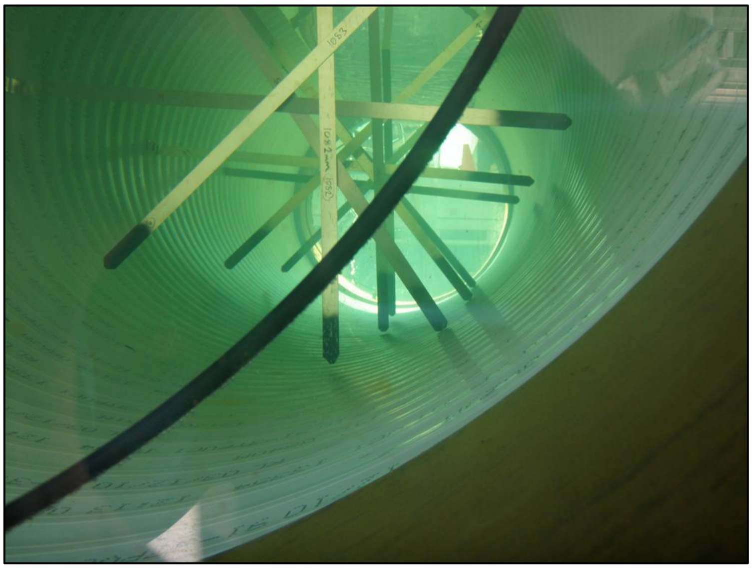
6. The space between the outside of the liner and inside of the pressure vessel was hydrostatically pressurised.

Pressure was increased until deformation in the liner reached a stage where the inner surface just touched an indicator bar. This occurred at a pressure of 30 kPa.

Picture 1: Assembly for the unsupported liner test, prior to placing the end sealing bungs. The Rotaloc liner is located centrally in the cylindrical pressure vessel

Picture 2: Installing the sealing bungs at the end of the liner

Picture 3: The inside surface of the liner is just touching the indicator bars, showing that the liner has deflected by 3%. For the purposes of the test, this was taken to indicate that the liner had buckled.



It should be noted that the aim of this test was not to determine the ultimate buckling capacity of the liner. The aim was to test until buckling had reached a particular point, well before buckling failure. The testing could then be repeated for the same liner in confined and grouted conditions, and measurement made of the loads needed for the liner to reach the same point. The difference in load could then be used to calculate the enhancement provided by the confinement and buckling.

By back-calculating, applying 30 kPa as the value for "q" in Equation 2 (above), with N = 1 and K = 1 (as it was unsupported), it can be determined that the appropriate value of "C" was 0.69. This equated to an ovality of 4.2%.

It could be surmised from the "C" value obtained, that approximately 30% of additional load would need to be applied to reach full buckling of the liner. This is in the order of what would be expected and so provided confidence that the test method was valid.

The precise test method could then be repeated identically until 3% deflection was reached, with the value of C = 0.69 used in calculation to determine values of the Enhancement Factor "K" in Equation 4 (above).

3.2.3 TESTING THE SUPPORTED, UNGROUTED LINER

A steel tube with an internal diameter of 1,200mm, composed of 6mm steel plate was fabricated in a length of 6 metres. Slots were cut into the tube running longitudinally and spaced around its circumference. (See Picture 4)

The tube was required for use in both the ungrouted and grouted liner testing, with the slots allowing hydrostatic pressure to be applied to the liner.

The inside of the steel tube was heavily greased with a de-bonding agent and tested to confirm that grout would not bond to it.

The Rotaloc liner was wound into the tube, fitting tightly against its inner surface. The tube with the Rotaloc liner inside was inserted into the cylindrical pressure vessel, and centred with wooden rails. (See Picture 5)

The ends of the annulus between the outside of the steel tube and the inside of the pressure vessel were then sealed, and hydrostatic pressure applied.

The inside of the Rotaloc liner touched the 3% deflection indicators at an external hydrostatic pressure of 150 kPa, indicating that the support provided to the liner increased its load carrying capacity by a factor of five (K = 5 in Equations 1 and 2, above)

This compares with a value of C = 4 typically specified for Equation 2 by Australian and New Zealand Water Authority liner design Specifications.

Picture 4: Steel tube fabricated to support the Rotaloc liner to be wound inside it. Slots were cut in the tube to allow hydrostatic pressure to be applied to the liner.

3.2.4 TESTING THE SUPPORTED, GROUTED LINER

Following the testing of the ungrouted liner, the steel tube was removed and the slots covered.

The ends of the liner were sealed with expanding foam and the liner grouted with the normal type of grout and by the normal process used by Interflow to grout spiral wound liners.

Following curing of the grout, the covers over the slots were removed and the liner and its steel tube were re-inserted and re-centred in the pressure vessel. Removing the covers from the slots was necessary to allow the hydrostatic pressure to be applied to the grouted liner.

The annulus around the steel tube was then sealed at the ends and pressure applied.

The maximum hydrostatic pressure available from the pump was 600 kPa. This pressure was reached with no evidence of any deformation of the liner. It was obvious that the pressure carrying capacity of the grouted liner was considerably greater than 600 kPa.

Nevertheless this test showed that grouting of the liner increased its load carrying capacity by a factor greater than 4 compared to an ungrouted liner, and by a factor greater than 20 compared to an unsupported liner.

This factor of 20, which can be applied as the “C” value in Equations (2) and (4), above, compares with a value of 7 for grouted liners commonly specified by Australian and New Zealand Water Authorities.

Picture 6: Slots in the steel tube covered while the liner is grouted. The covers were removed prior to it all being inserted into the pressure vessel for testing.

4 CONCLUSIONS

This testing has shown that grouted spiral wound liners have much greater load carrying capacity than has previously been assumed by commonly accepted liner design Specifications.

In summary the testing has returned the following results.

Table 2: "K" Factors from Testing

Liner Support	**Test load carrying capacity**	**"K" from testing**	**Current "K" design value**
Unsupported	30 kN/m^2	1	1
Supported by host pipe	150 kN/m^2	5	4
Supported by host pipe and grouted	+600 kN/m^2	+20	7

The result obtained for the ungrouted liner is consistent with results from previous testing carried out by others on spiral wound and CIPP liners. This provides confidence that the testing method is valid.

It is understood that this is the first comprehensive testing carried out on grouted spiral wound liners. While the magnitude of the result obtained could not be confidently predicted prior to testing, the "K" factor obtained is not unexpected. Grouting can be considered to not only support the spiral wound liner, but also provide it with increased stiffness because of the grout filling in and supporting the tees.

While the testing did not expose the limit of the liner support value provided by grouting because of the limitations of the test equipment, using the value of enhancement factor of K = 20 means that it is valid to conclude that a grouted liner can support far greater loads than previously considered.

The implications of these results are far reaching, particularly for large diameter sewer and stormwater pipelines. While it can result in significant reductions in the cost of pipeline rehabilitation, it makes lining possible in many circumstances where current systems are not able to meet the required stiffness. There are many markets and applications where a change in the cost and engineering dynamics mean the difference between whether or not a viable rehabilitation solution can be provided.