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Structural Rehabilitation of Cast-In-Place Concrete Sewers

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Abstract

This paper discusses the design and installation of a Grouted-In-Place PVC liner (GIPL) in a large diameter cast-in-place concrete pipe (CIPCP). The Sims Bayou trunk sewer in southeast Houston, Texas consists of several miles of 72" (1.83 m) and 78" (1.98 m) nominally circular concrete structures built about 40-years ago. This paper deals with the structural rehabilitation of approximately 1500 feet (457 m) of 72" (1.83 m) and 8500 feet (2,591 m) of 78" (1.98 m) of this sewer that has deteriorated due to hydrogen sulfide induced corrosion of the inner surfaces of the pipe walls. The design of the GIPL follows classical engineering mechanics for composite reinforced concrete structures. Design examples are given for various levels of deterioration of the host structure.

Introduction

In June 2004 the city of Houston awarded two contracts for Package 2 and Package 3 for the Sims Bayou Large Diameter Pipe Rehabilitation Project to Boyer, Inc. of Houston, Texas to install the Danby GIPL (ASTM F 1698). Package 2 involves the rehabilitation of approximately 1500 feet (457 m) of 72" (1.83 m) ID and 3500 feet (1,067 m) of 78" (1.98 m) ID CIPCP and Package 3 covers 5000 feet (1,524 m) of 78" (1.98 m) ID CIPCP. Characteristically for CIPCP the actual ID varies significantly from the nominal value as do both horizontal and vertical alignments. The bid specifications allowed a choice of cured-in-place-plastic (CIPP) liner, grouted-in-place-PVC (GIPL) liner and fiber reinforced plastic (FRP) sliplining all requiring minimum IDs of 6" less than the host pipe ID. Hydrogen sulfide induced corrosion varied from moderate to severe producing areas of exposed rebar reinforcement with levels of rebar steel cross-sectional area loss from none to severe. However, there were no collapsed areas and very few areas with longitudinal cracks. The liner design assumes that the rehabilitated pipe carries the total soil and hydrostatic load with a safety factor of 2.0.

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GIPL Installation

The three major work processes involved in the installation of the GIPL are cleaning of the pipe wall, spiral winding of the PVC lining into a tube with an inner diameter about 4" (101.6 mm) less than that of the host pipe and grouting the annular space between the PVC liner and the host pipe. In any given section of pipe the work proceeds in the order as listed, i.e., cleaning, lining, grouting. However, on a large (length) project such as this, three work crews can execute these three tasks simultaneously. Of course, there are several other subsidiary tasks such as lateral and side sewer reconnection and building of bulkheads at manholes and intermediate bulkheads that serve as grout dams.

***Cleaning*.** The three principal stages of cleaning of large diameter pipes are 1) removing loose debris from the invert, 2) removing loosely bonded deposits from the pipe wall and 3) removing more tightly bonded deposits and hydrogen sulfide induced concrete corrosion products (principally from the upper regions of the pipe wall). Smaller volumes of invert debris can be removed by high-pressure water flushing to a manhole followed by either vacuum or mechanical removal from the pipe. The level of debris in the invert of the Sims pipes required dragging a specially designed bucket, shaped to match the pipe invert, through the pipe to effect mechanical removal of the bulk of debris. The second stage cleaning was accomplished by the innovative use of large construction equipment pneumatic tires as huge squeegees/swabs (see Figure 1). Hydroblasting (10,000 psi (69 MPa)) final stage cleaning removed all remaining material leaving only competent concrete. In the upper reaches of the pipe walls, where the H2S corrosion was prominent, the surface is very rough with well-anchored large aggregate exposed (See Figure 2 and Figure 3).

Figure 1. Cleaning bucket and tire swabs.

Cleaning of the pipe wall down to competent concrete is important to the structural repair of the deteriorated host pipe. The design requires adequate bonding of the grout to the pipe wall to insure composite material action (response to loads). Many studies have shown (WRc 1983, Ahmad and McAlpine 1994) that the

Figure 2. Host Pipe (after cleaning).

composite action can be achieved with fairly low bond strength of the grout to the pipe. The essential requirement is that all strains (circumferential movement) in the pipe wall are transferred to the grout, producing equal strains in the grout. The rough surface of the pipe wall after cleaning, with protruding aggregates, insures that the grout is, at a minimum, mechanically bonded to the pipe wall. This mechanical lock of the grout with the pipe wall is sufficient to insure the transfer of circumferential strains. Further, various field tests (Ahmad and McAlpine 1994) have shown that significant bonding of the grout to the concrete (direct tension pull test) does occur. As corrosion levels dictate, steel welded wire mesh is placed to cover areas of corroded steel reinforcement (rebar). See Figure 3 and the Appendix *Sample Design Calculations*.

Figure 3. Added steel reinforcement and hydroblasted surface

PVC Liner Installation**.** The PVC liner was delivered to the job site in 300 ft (91 m) coils (ID = 64 in (1.63 m), OD = 90 in (2.3 m)). The 12 in (305 mm) wide profiled liner has "T" shaped ribs on one side (overall height of 1.0 in (25.4mm)) and a smooth surface on the other (flow surface). The profile has a nominal moment of inertia of 0.015 in^4/in (246 mm^4/mm). The PVC is ASTM D 1784 cell class 13343 with nominal flexural modulus of 430,000 psi (2,966 MPa). These physical properties provide a hydrostatic buckling strength of over 90-ft (27.4 m) of water head in a 6 in (152 mm) arc of radius 37 in (940 mm) (ASTM F1698, Appendix 1). This implies that the ungrouted liner arc of 6 in (152 mm) would withstand the Sims project ground water head of 7 ft (2.1 m) with a safety factor of more than 12.

The liner was taken into the pipe's interior by simply pulling the PVC liner from the inside of the bound (at the OD) coil down through the manhole. The uncoiled liner in the pipe invert appears to be a long "Slinky". One end of the liner (usually at the upstream starting point) was formed into a circular hoop of the desired diameter and the edge joints of adjacent windings joined together by a second "joiner" strip that was inserted with an air hammer. The joiner strip has a co-extruded rubber gasket that forms a compression seal making the joint water and gas tight. This process of spiral winding of the PVC liner continued (moving toward the unwound "Slinky") until the full 300 ft (91 m) of liner was installed (Figure 4). The next coil was joined to its predecessor with a short length of "H-Section" that was sealed with a suitable caulking compound (ASTM F1698). As the spiral winding proceeded past laterals or side sewers, holes were cut into the liner and PVC pipes/tubes were inserted into the lateral and through the PVC liner. The new inserted PVC tube/pipe was sealed both in the existing lateral/side sewer and at the PVC liner surface before the annulus grouting reached it to prevent grout filling the lateral/side sewer and or leaking into the interior of the PVC liner.

Figure 4. Installed GIPL and grout dam/bulkhead.

In all such installations, it is very important that the upstream end of the liner have a very solid and stable bulkhead built into the annulus between the pipe wall and the PVC liner to prevent any sewerage flows from getting behind the liner. Even an ungrouted liner can handle significant flows as long as it is restricted to the interior of the liner. These bulkheads are usually made of quick setting hydraulic cement packed into the annulus behind the liner. Similar bulkheads are required at every break in the continuous liner, e.g., if the lining does not extend through intermediate manholes. In this project, the contractor chose to line continuously through all manholes and provide access by cutting a suitable hole through the top of the liner. Of course, suitable bulkheads must be installed around all such cuts in the liner to prevent annulus grout from entering the liner's interior. Sims project specifications requires 18 new manholes with Ameron T-Lock™ lined 6 ft (1.83 m) diameter concrete risers. The (PVC) T-Lock™ riser liners are welded to the Danby PVC liner to produce an entirely PVC sewer interior surface that will be impervious to the hydrogen sulfide corrosive effects.

Annulus Grouting. All structural rehabilitation of the host rigid pipe by GIPL is due to the effects of the high strength annulus grout, in conjunction with added steel if used. The PVC liner provides a smooth flow surface, a chemically protective containment, and formwork for the annulus grout. Its contribution to structural improvement is not significant. Thus the composition and placement of the annulus grout is very important to the proper installation of any GIPL.

The properties of a suitable annulus grout are high strength (e.g., 5,000 psi (34.5 MPa) compressive strength), low viscosity (flow cone time less than 35 sec.), low shrinkage (less than 1%), and non segregating over long flow distances in small spaces. Another important characteristic is that the grout mix design can produce highly repeatable results over thousands of individual batches of grout.

Figure 5. Partially grouted liner at 72” (1.83 m) to 78” (1.98 m) transition.

In most applications a mix of Portland cement and fly ash in a low water-to-solids ratio combined with a superplasticizer and an anti shrink chemical admixture will produce a suitable grout. The contractor used such a mix design on this project with excellent results.

Repeatability of grout mixes is greatly influenced by consistent addition of the proper amount of mix water and chemical admixtures and consistent mechanical mixing. Automation removes human errors and improves repeatability. The mobile grout plant employed on this project had programmable logic controlled metering of the mix water and chemical admixtures as well as the mixing time of the colloidal mixer. It was completely self sufficient with its own power sources, water and chemical admixture tanks and PLC meters, colloidal mixer, separate mixing and pumping hoppers. This grout plant (see Figure 6) produced as much as 20 CY (15 m^3) of good quality grout in a single workday.

Because the PVC liner is a relatively low stiffness formwork the grout must be placed in lifts of limited vertical rise. Typically, the first lift is limited by flotation of the liner while later lifts are limited by the need to prevent the grout pressure (hydrostatic head of the liquid grout) forcing the PVC liner out the previous lift. The pressure required to break the liner out of the previous lift is determined by the strength of the curing grout. Assuming a compressive strength of 1,000 psi (6.9 MPa) at the time of pouring the next lift, the lift height is limited to about 10 in.

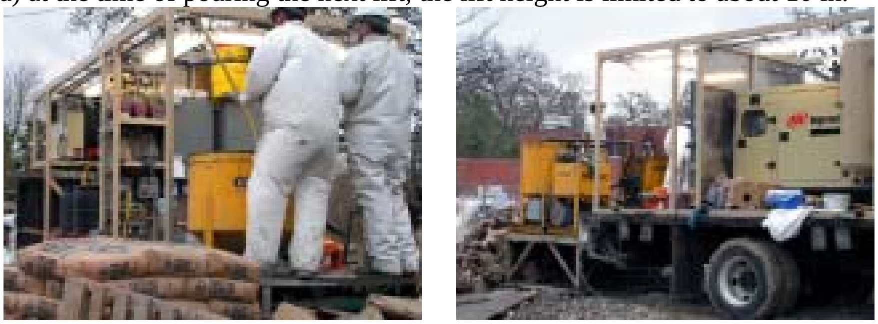

Figure 6. Mobile grout plant

(254 mm). Compressive strength tests at 24 hours indicate grout strength of 1,000 to 3,000 psi (6.9 to 20.7 MPa) for the Sims project grout. This analytical estimate is very conservative and actual job experiences indicate that larger grout lift heights could be employed safely. Drilling holes in the liner at the desired height can control the lift heights. Grouting was stopped when grout reached the level of the indicator holes. These holes were then sealed with special PVC grout plugs that were also used in the holes in the liner for introducing the grout and for allowing annulus air to escape as the grout was introduced into the annulus. Annulus bulkheads (Figure 4) were built at predetermined distances that serve as grout dams to facilitate more accurate estimates of the amount of grout required to place a given lift of grout.

GIPL Design

Before discussing the design of GIPL liners for the rehabilitation of rigid pipes in general, and reinforced concrete pipes in particular, it is appropriate to discuss the design of reinforced concrete pipe (RCP) and the design practices of installing manufactured RCP. Although the Sims project pipes are CIPCP (or sometimes-called MRC for monolithic reinforced concrete), the same design principals apply. In addition, the characteristics and design parameters of manufactured RCP are more accessible from such widely available sources as ASTM C 76.

Because it is practically impossible to bury a RCP such that the soil load is uniform and perfectly radial, the failure mode of buried RCP is cracking due to bending moments created by asymmetrical vertical and horizontal soil pressures (typical horizontal-to-vertical ratio of 0.4). Thus the appropriate RCP attribute to specify and test is its resistance to cracking. This cracking strength (for both unreinforced concrete pipe and RCP) is called “D-load” strength that is determined by the “Three Edge Bearing” test or TEB test (Moser 1990, page 44). The TEB test places the pipe under increasing vertical (hydraulically applied) line load at the top of the pipe and resisted at the bottom of the pipe by two closely space knife edge supports. This produces almost pure bending with maximum positive moments at the top and bottom and the maximum negative moments at the springlines. These moments produce maximum flexural stresses (mostly tensile) on the inner surface of the pipe at the crown and invert and on the outer surface at the springlines. The “D-load” strength is defined as the ratio of the vertical load per foot required to produce a crack of 0.01 in (0.254 mm) (an arbitrary value, but now standard) divided by the internal diameter (in feet) of the pipe.

The TEB test is continued, after cracking, until the pipe deflection increases without increasing vertical load. The vertical load at which this first occurs is termed the “Ultimate” load and typically measures about 150% of the D-load value for RCP and 100% for unreinforced concrete pipe. Because RCP has significantly greater load bearing capacity than indicated by its D-load strength, failure strength is measured by the Ultimate strength. Thus, the indirect design method for concrete pipes recommends a safety factor of 1.0 for RCP and 1.25 – 1.5 for unreinforced concrete pipe where the SF is the ratio of D-load strength to effective soil load (Moser 1990, page 46). The effective soil load is the vertical prism soil load divided by a bedding factor (BF). The BF (>1.0) accounts for the lower bending moments resulting from good bedding and side soil support compared to that of the TEB test.

The industry practice of using SF = 1.0 for RCP reflects the acceptance of the fact that concrete cracking is not to be considered failure of the pipe nor does it define the upper limit of the RCP ability to carry its structural load. In fact, a buried concrete pipe with good soil support will behave (load versus deflection) as a high-stiffness flexible pipe once the load exceeds the pipe’s ultimate strength. (Actually, the pipe alone has zero stiffness but the pipe-soil structure has a stiffness determined by the soil’s modulus of soil reaction.) (Watkins, 1993). A detailed analysis of the RCP in bending will show that the steel cannot carry significant loads until the concrete cracks. This is because the two materials are not strain compatible, i.e., steel’s elastic modulus is 10-20 times that of concrete and concrete is strain limited. The strain limit of concrete is between 100 – 200 microstrains (it then cracks). Before cracking the strains in the concrete and steel are approximately equal, resulting in a steel stress of only about 5% - 10% of its yield stress. Thus, the concrete must crack before the steel can carry any significant load. Therefore, the concrete principally determines the D-load strength and the steel principally determines the ultimate strength. This can be confirmed by analyzing the strengths of RCP given in ASTM C76 (also see Watkins and Anderson 1999).

This same type of analysis and reasoning applies to the low modulus (10% relative to concrete) polymer linings such as the PVC used in the subject GIPL. At 200 microstrains the stress in a PVC liner fully bonded to the concrete will only be about 1.3% of its tensile yield strength. However, unlike steel, it offers little resistance to further strain (modulus of 1% of steel and tensile strength of only about 10% of steel). Thus it would require very thick polymer liners bonded to the concrete to have any significant impact on structural performance of deteriorated RCP. On the other hand, grout bonded to the RCP pipe wall does enhance the structural performance of the deteriorated RCP because its physical properties are very similar to the concrete in the pipe wall. The modulus and strength of the grout used in the Sims GIPL is equal to or superior to that of the host pipe concrete. Further, unlike both steel and PVC the grout and concrete are strain compatible, having approximately equal strain limits. This means that the grout does enhance the pipe’s cracking strength by increasing the wall section modulus (about 60% for this project). Of course steel must be added to significantly enhance the pipe's ultimate strength.

Conclusions

This paper has presented a real life case study of the structural rehabilitation of a deteriorated pipeline constructed as a cast-in-place reinforced concrete pipe by a grouted-in-pace liner (GIPL). The GIPL installation process was described in some detail and the design principles were outlined. Although the authors have attempted to give a lucid account of the design theory and its relation to classical engineering mechanics for composite structures, it has been largely descriptive and non analytical. The Appendix, Sample Calculations, adds specificity and details the mathematical steps involved in the design process. Further, it verifies, numerically, the capacity of the GIPL process to enhance the structural capacity of a host reinforced concrete pipe. While these calculations are routine for structural engineers designing reinforced concrete structures following ACI 318, this body of knowledge and literature is largely absent in the pipe rehabilitation industry.

Appendix: Sample Calculations

- Original design: 78" (1.98 m) I.D. monolithic reinforced concrete (MRC) sewer.
- Sketch shows original dimensions of MRC sewer as shown in as-built drawings.
- Use Factor of Safety of 2.0 against ultimate failure using ultimate strength theory (per project specifications).
- Check service load stresses.

Codes and References:

- Danby Pipe Renovation Design Guide.
- Concrete Pipe Design Manual, ACPA.
- PCI Design Handbook, 5th Ed.
- ACI 318-02, Building Code Requirements for Structural Concrete.

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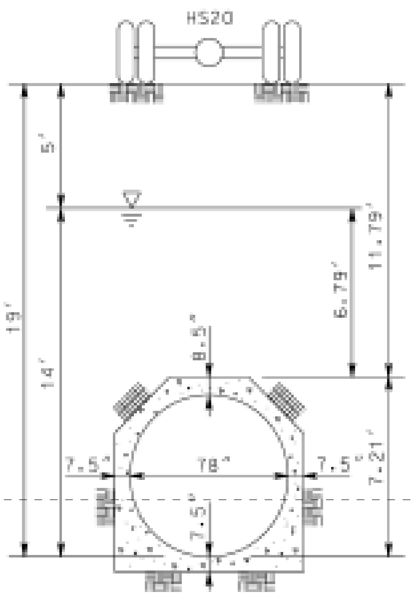

Figure A1. Original cross-section.

Design Loads:

- HS20 Truck, no impact (H>3', H>0.91 m).
- 120 lbs/ft^3 (1922 kg/m^3) soil density.
- Hydrostatic: water table 5 ft. (1.52 m) below surface.

Vertical Load on Structure

Per Danby Design Guide:

q_t = 62.4 lbs/ft^3(H_w) + γ_s(H)(R_w) + WL + W_{sc} (total vertical pressure)
<u>Hydrostatic Load</u>: 62.4 lbs/ft^3(6.79') = <u>424 lbs/ft^2</u> (20.3 kPa)

<u>Soil Load</u>: (Rw) = 1-0.33(H_w/H) = 1-0.33(6.79'/11.79') = 0.81
γ_s(H)(Rw) = 120 lbs/ft^3(11.79')(0.81) = <u>1146 lbs/ft^2</u> (54.9 kPa)

<u>HS20 Wheel Load</u>: WL = P(1+I_R)/A_{LL}
Per American Concrete Pipe Association's Concrete Pipe Design Manual
for H > = 4.1':

P = 48,000 lbs A_{LL} = [4.83+1.75H][5.67+1.75H]
= [4.83+1.75(11.79')][5.67+1.75(11.79')] = 670 ft^2 (62.24 m^2)

WL = 48,000 lbs/ 670 ft^2 = <u>72 lbs/ft^2</u> (3.45 kPa)

<u>Surcharge Load</u>: No additional surcharge, therefore W_{SC} = 0
<u>Total Load</u>: q_t = 424 lbs/ft^2 + 1146 lbs/ft^2 + 72 lbs/ft^2 = <u>1642 lbs/ft^2</u> (78.6 kPa)

<u>Analyze Section at Crown</u>

<u>Case 1</u>
Assumptions:
- 2" of concrete corrosion.
- Original reinforcement still intact.
  No cross-section loss due to corrosion.

<u>Check Stresses</u>

A_S = 0.2 in.^{2} /(6.5"/12in./ft.) = 0.369 in^2/ft.

f_y = 40,000 psi (276 MPa)

f'_c = 3000 psi (20.7 MPa)

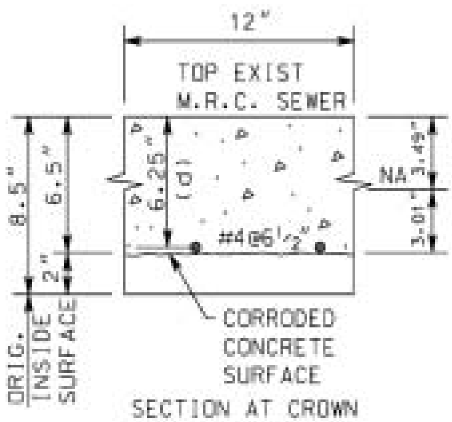

Figure A2.

Transformed Section:
- Use long term value for modulus of elasticity for the concrete (1/2 short term).

E_c = 0.5(145pcf)$^{1.5}$(33)(3000 psi)$^{0.5}$ /1000 lb/k = 1578 ksi (10,880 Mpa)

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E_s = 29,000 ksi (199,955 Mpa)
modular ratio, n = 29,000 ksi / 1578 ksi = <u>18.4</u>

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Neutral Axis, $Y_{NA} = \dfrac{6.5''(12'')(3.25'')+18.4(0.369\text{ in.}^2)(0.25'')}{6.5''\,(12'')+18.4(0.369\text{ in.}^2)}$ = <u>3.01"</u> (77 mm)

Transformed Moment of Inertia:

$I_{TR} = \Sigma\, I'_x$, $I'_x = n_x I_x + (Y_{NA} - Y_x)^2 n_x A_x$

I_{TR} = (1/12)(12")(6.5")^3+(3.01"-3.25")^2(12")(6.5")
+(3.01"-0.25")^2(18.4)(0.369in.^{2}) = 275 in.^{4} + 5 in.^{4} + 52 in.^{4} = <u>332 in.^{4}</u>

Moment at Crown:

M_c = [q_t(OD)r/8] [C^2 - K(C')^2] q_t = 1642 lbs/ft^2/144 in.^{2}/ft.^{2} = 11.4 psi (78.6 kPa)
(per Danby Design Guide)
- Outside diam. based on round pipe w/ 8.5" (216 mm)wall thick and 78" (1.98 m) I.D.
OD = 78" + 2(8.5") = 95" (2.41 m)
r = (78"/2)+2"+(6.5"/2) = 44.25" (1.12 m) (mid-thickness radius of deteriorated pipe)
K = 0.4 (Coefficient of active earth pressure, assumed)
C = 1+0.002 = 1.002 C' = 1-0.002 = 0.998

M_c = [11.40 psi(95")(44.25)/8] [(1.002)^2 - 0.4(0.998)^2] = <u>3628 lb.-in./in.</u>
Over 12" (305 mm) section: M_c = 3628 lb.-in./in.(12") = <u>43,536 lb.-in.</u> (4919 kN-mm)

Thrust at Crown:

T_h = Kq_tr / T = 0.4(11.40 psi)(44.25") / 6.5" = <u>31 psi</u> (214 kPa)
(T is corroded wall thickness)

<u>Tensile Stress In Concrete at Crown</u>

Cracking stress: f_{cr} = 7.5(3000 psi)$^{0.5}$ = 411 psi (2.83 MPa)

$\sigma_{T,C}$ = M_cc / I_{TR} - T_h = 43,536 lb.-in.(3.01") / 332in.^{4} - 31 psi = <u>364 psi O.K.</u>
(2.51 Mpa)

<u>Tensile Stress in Steel at Crown</u>

$\sigma_{T,S}$ = nM_cc / I_{TR} - nT_h = 18.4(43,536 lb.-in.)(2.76") / 332in.^{4} - 18.4(31 psi)
= <u>6,089 psi O.K.</u> by inspection w/ fy = 40,000 psi (276 Mpa)
(42 MPa)

<u>Compressive Stress in Concrete at Crown</u>

$\sigma_{C,C}$ = M_cc / I_{TR} + T_h = 43,536 lb.-in.(3.49") / 332in.^{4} + 31 psi = 489 psi O.K.
By inspection w/ f'_c = 3000 psi(20.7 Mpa) (3.37 Mpa)

<u>Check Ultimate Strength at Crown</u>

- Ultimate strength of section must provide a factor of safety of 2 against failure.

M_n = $A_s f_y$ [d - 0.59 ($A_s f_y$) / (f'_c b)]
= 0.369 in.^{2}(40 ksi) [6.25" - ((0.59)(0.369 in.^{2})(40 ksi)) / (3 ksi(12"))]

= <u>88.7 kip- in.</u> (10,023 kN-mm)

F.S. = 88.7 kip-in. / 43.5 kip-in.. = <u>2.04 > 2 O.K.</u>

<u>Case 2</u>

- Assume a 40% loss of cross-section for the reinforcing steel due to corrosion. Supplemental reinforcing steel will be required.
- Add (1) layer of WWF 6x6-W2.9xW2.9. Supp. reinf. steel will be placed near the top of the Danby liner at the crown.

$A_s = (1-0.4)(0.369\ in.^2) = 0.22\ in.^2$ $f_y = 40$ ksi

$A^*_s = 0.058\ in.^2$ $f^*_y = 80$ ksi

$M_n = A_s f_y(d-(a/2)) + A^*_s f^*_y(d^*-(a/2))$

$a = (A_s f_y + A^*_s f^*_y) / (0.85(f'_c)(b))$

$= \dfrac{0.22\ in.^2(40\ ksi) + 0.058\ in.^2(80\ ksi)}{0.85(3\ ksi)(12")}$

$= 0.44"$ $a/2 = 0.22"$

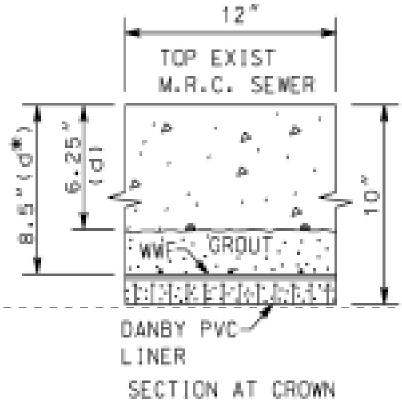

Figure A3.

$M_n = 0.22\ in.^2(40\ ksi)(6.25"-0.22") + 0.058\ in.^2(80\ ksi)(8.5"-0.22")$

$= 53.1$ kip-in. + 38.4 kip-in. = <u>91.5 kip-in.</u>

(10,340 kN-mm)

<u>Analyze Section at Springline</u>

Assumptions:
- 1" of concrete corrosion.
- Original reinforcement still intact. No cross-section loss due to corrosion.

<u>Check Stresses</u>

$A_S = 0.2\ in.^2 /(6.5"/12 in./ft.) = 0.369\ in^2/ft.$

$f_y = 40{,}000$ psi (276 Mpa) $f_c = 3000$ psi (20.7 Mpa)

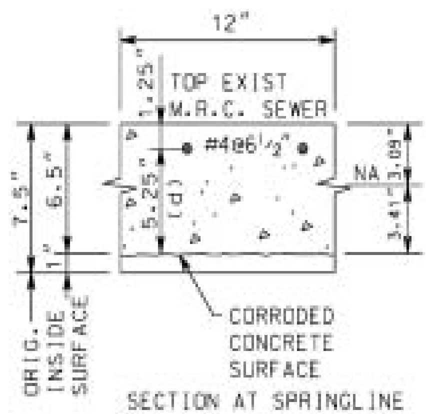

Figure A4.

Transformed Section:

Neutral Axis, $Y_{NA} = \dfrac{6.5"(12")(3.25")+18.4(0.369\ in.^2)(5.25")}{6.5"(12")+18.4(0.369\ in.2)} = \underline{3.41"}$ (87 mm)

$I_{TR} = (1/12)(12")(6.5")^3+(3.41"-3.25")^2(12")(6.5")$
$+(3.41"-5.25")^2(18.4)(0.369in.^2) = 275\ in.^4 + 2\ in.^4 + 23\ in.^4 = \underline{300\ in.^4}$

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Moment at Springline:

$M_{SL} = [q_t(OD)r/8]\ [\ K(C')^2 - C^2\ ]$
Moment at Springline Continued:

- Outside dia. based on round pipe w/ 7.5" (191 mm) wall thick and 78" (1.98 m)I.D.
  OD = 78" + 2(7.5") = 93" (2.36 m)

r = (78"/2)+1"+(6.5"/2) = 43.25" (1.10 m) (mid-thickness radius of deteriorated pipe)

M_{SL} = [11.40 psi(93")(43.25)/8] [0.4(0.998)^2 - (1.002)^2] = <u>3472 lb.-in./in.</u>

Over 12" (305 mm) section: M_{SL} = 3472 lb.-in./in.(12") = <u>41,664 lb.-in.</u> (4708 kN-mm)

Thrust at Springline:

$T_h = q_t r / T$ = 11.40 psi(43.25") / 6.5" = <u>76 psi</u> (524 kPa)
(T is corroded wall thickness)

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<u>Tensile Stress In Concrete at Springline</u>

Cracking stress: f_{cr} = 411 psi (2.83 Mpa)

$\sigma_{T,C} = M_{SL}c / I_{TR} - T_h$ = 41,664 lb.-in.(3.09") / 300in.^{4} - 76 psi = <u>353 psi O.K.</u>
(2.43 Mpa)

<u>Tensile Stress in Steel at Springline</u>

$\sigma_{T,S} = nM_{SL}c / I_{TR} - nT_h$ = 18.4(41,664 lb.-in.)(1.84") / 300in.^{4} - 18.4(76 psi)
= 3,304 psi (22.8 Mpa) O.K. by inspection w/ fy = 40,000 psi (276 Mpa)

<u>Compressive Stress in Concrete at Crown</u>

$\sigma_{C,C} = M_{SL}c / I_{TR} + T_h$ = 41,664 lb.-in.(3.41") / 300in.^{4} + 76 psi = <u>550 psi O.K.</u>
By inspection w/ f'_c = 3000 psi(20.7 Mpa) (3.79 Mpa)

<u>Check Ultimate Strength at Springline</u>

- Ultimate strength of section must provide a factor of safety of 2 against failure.

$M_n = A_s f_y\ [d - 0.59\ (A_s f_y) / (f'_c\ b)]$
= 0.369 in.^{2}(40 ksi) [5.25" - ((0.59)(0.369 in.^{2})(40 ksi)) / (3 ksi(12"))]
= <u>73.9 kip- in.</u> (8351 kN-mm)

F.S. = 73.9 kip-in. / 41.7 kip.in. = <u>1.77 < 2 NOT O.K.</u>

- Check section after adding 1.5" (38 mm)of grout on the inside of the sewer.

M_n = 0.369 in.^{2}(40 ksi) [6.75" - ((0.59)(0.369 in.^{2})(40 ksi)) / (3 ksi(12"))]

= <u>96.1 kip- in.</u> (10,859 kN-mm)

F.S. = 96.1 kip-in. / 41.7 kip.in. = <u>2.30 > 2 O.K.</u>

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