A 144- inch-Diameter Composite Steel-and-Concrete Tunnel Liner Designed For External Heads Up to 1200 feet

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Abstract: The 144-inch-diameter composite-steel-and-concrete tunnel liner for the Arrowhead tunnels below the San Bernardino mountain range in Southern California (the cover over the crown of the tunnel is as much as 1590 feet), 9 miles long, was designed for external heads up to 1200 feet (520 psi) and maximum internal pressure of 80 psi. The steel cylinders resist the internal pressure and a portion of the external pressure while the concrete cores resist the major portion of the external pressure.

The special liner design required high concrete compressive strength with high modulus of elasticity and low creep properties to ensure that there would be no overstress in the steel cylinder for the long term conditions. The composite-wall tunnel liner also required concrete core reinforcement designs to withstand all pipe handling and installation forces.

Structural analyses of the pipe, with a few preliminary reinforcement schemes including embedded studs, were conducted assuming a line-bearing support with 50% impact. The analysis included calculations for maximum pipe deflections and maximum stresses due to moment and thrust at the pipe invert and springline. Shear and radial tension was also checked. Two test pipe sections were manufactured (based on the final proposed reinforcement scheme) and were subjected to load-deflection tests to verify the pipe design.

In this paper, the overall design concept, the summary of the structural analyses results, creep and modulus qualification tests, pipe-deflection test results, and the steel cylinder fabrication testing requirement will be presented. The advantages of this unique concept tunnel-liner design for high external heads will be discussed.

Figure 1. 144-in. Diameter Composite Wall Tunnel Liner

Introduction

The Arrowhead tunnels, up to 1590 feet below the San Bernardino mountain range, is part of the Metropolitan Water District of Southern California's Inland Feeder pipeline which will double the water delivery capacity from the California State Aqueduct to Southern California (The Colorado River Aqueduct and Diamond Valley Lake). The 144-inch diameter Arrowhead tunnels consist of the Arrowhead East and Arrowhead West tunnels which are approximately 30,000 feet long and 20,000 feet long, respectively. The tunnels were designed with a two-pass liner system. The initial precast reinforced concrete segmental tunnel liner with gasketed connections is designed to withstand all ground loads and water infiltration during construction and the final liner is designed to withstand the internal and external pressures.

Two alternative final tunnel liner concepts were considered. One alternative was a flexible steel tunnel liner with both stiffened and un-stiffened options designed to withstand the internal pressures and high external heads. The second option was a rigid-wall composite steel-and-concrete tunnel liner designed to withstand the internal pressures and high external heads. All project bidders for both tunnels choose the second option. Therefore, only the second option will be addressed in this paper. One pipe section manufactured in conformance with this composite wall option is shown in Figure 1.

The design concept of the rigid-wall composite steel-and-concrete tunnel liner is based on having the concrete core resisting more than 60% of the external hydrostatic pressure and the steel cylinder resisting the remainder of the external pressure and 100% of the internal pressure. The special liner design required high compressive strength, a low creep factor, and high modulus values.

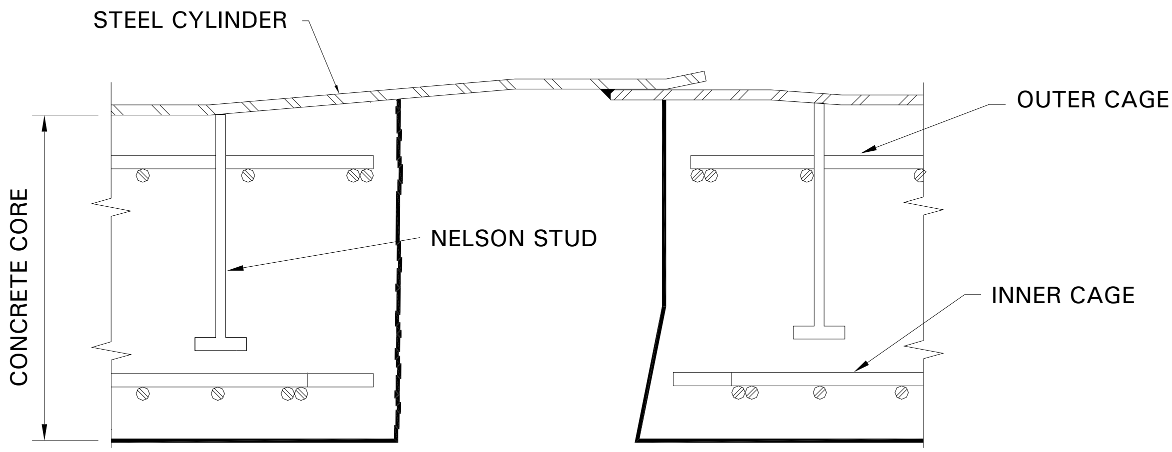

Figure 2. Components of the Composite Wall Tunnel Liner

The main challenges are the selection of aggregates; conducting qualification creep and modulus testing; and the design and testing of the concrete core reinforcement to ensure that the structural integrity of the composite wall design, during pipe handling and installation, is not compromised.

Design Concept of the Composite Wall Tunnel Liner

The steel cylinder shown in Figures 1 and 2 resists the internal pressure. The internal design pressure varies between approximately 60 psi and 80 psi. The specified minimum yield strength, σ_y, and tensile strength, σ_T, of the steel cylinder are 42,000 psi and 70,000 psi, respectively. The minimum cylinder thickness specified is 0.3125 inches. Therefore, based on the hoop stress formula, the specified minimum cylinder thickness can withstand an internal pressure of 80 psi, assuming an inside diameter of 164 inches and a design stress of 21,000 psi (50% of $\sigma_y = 42{,}000$ psi).

The Metropolitan Water District of Southern California's consultant designed the liner for an external hydrostatic pressure varying between 300 feet and 1200 feet for the Arrowhead East Tunnel, and varying between zero and 800 feet for the Arrowhead West Tunnel. For analysis, the composite wall was transformed into an equivalent concrete section which resists the external pressure P. The concrete stress and strain of the transformed section due to P/A, where A is the transformed area, is reduced after creep in the concrete core. The decrease of concrete strain due to creep results in an equal increase in the steel cylinder; therefore the maximum creep in the concrete core was controlled to avoid overstressing the steel cylinder. The required creep factor at time of external hydrostatic loading (assuming a minimum of 60 days after manufacture) was a maximum of 0.85.

The design concrete core thicknesses are 9 in. for external heads ≤ 400 ft., 10 in. for external heads 500 ft. to 600 ft., and 12 in. for external heads ≥ 700 ft. The required 28-day concrete strength, f'_c, varied between 5000 psi and 7000 psi. The required modulus of elasticity of concrete, E_c, varied between 3.6×10^6 psi and 4.1×10^6 psi.

The analysis, design, and testing of the reinforcement of the concrete core and the shear connection between the cylinder and the core was the pipe manufacturer's responsibility.

Components of the Composite Liner

As shown in Figure 2 the tunnel liner consists of a steel cylinder with thicknesses varying from 0.3125 inches up to 0.75 inches and 10 inch or 12 inch thick reinforced concrete core. The field joint is a lap welded joint for steel cylinder thicknesses up to 0.375 inches and butt welded joints for thicker cylinders; only the lap welded joint is shown in Figure 2. The back up bars for the butt welded joint can be observed in the picture of Figure 1.

Testing Requirements for the Steel cylinder

All steel cylinders were hydrostatically tested to 120 psi. In addition to the destructive testing requirements of the weld seams in conformance with the AWWA C200 standard, 100% of the weld seams were radiographically tested to ensure the watertightness of the steel cylinders.

Concrete Creep and Modulus Qualification Tests

Regular creep and shrinkage tests in conformance with ACI 209 are usually conducted on 6 in. x 12 in. concrete cylinders loaded with 0.4 f'_c intensity, @ 72º F and 50% relative humidity. The loading age is usually 7 days for prestressed concrete members with tests completed in 37 days. However, for this project the loading age was going to be 60 days at least and it was also determined that creep was going to be measured for 90 days after the 60 day loading time. Therefore the duration of the test requires at least 150 days in addition to test preparation and report writing time. The pipe could not be manufactured until the creep test was successfully completed.

The first step was to identify the available sources of coarse and fine aggregates and conducting the 37 days creep and shrinkage tests for verification. Many sources of coarse and fine aggregates in Southern California were investigated. Five initial creep and shrinkage tests were conducted using the 7 day loading time to identify the best sources of aggregates. Compressive strength and modulus testing were also conducted concurrently with the creep and shrinkage tests. The best available coarse aggregate and fine aggregate source were identified after 8 months from start of testing; the coarse and fine aggregates were from two different sources.

The next step was to conduct the final creep and shrinkage tests on the proposed 6000 psi concrete with the candidate aggregates and cement. The average long term creep factor for the 6 inch x 12 in cylinders loaded and measured after 7 days, 28 days, 56 days, and 92 days, as determined from ACI 209, was 1.54. After making the other ACI 209 creep factor corrections for the loading age(60 days), relative humidity (95%) and volume to surface ratio (12 inch wall), the corrected creep factor became 0.65 which was less than the required 0.85.

In addition concrete compressive strength and modulus tests for both the 6000 psi and 7000 psi concrete were conducted concurrently with the creep and shrinkage tests; the compressive and modulus tests were conducted at 7 days, 28 days, 62 days, and 154 days. The summary of the average 5 specimen results is:

Mix Design	Age, days	Ave. Compressive Strength, psi	Average Moduli of Elasticity (10^6 psi)
6000 psi	7	5970	3.7
	28	6400	3.8
	62	7420	4.1
	154	8020	4.4
7000 psi	7	7250	3.8
	28	7750	4.1
	60	8150	4.3
	154	8500	4.5

The summary of the concrete compressive strength and modulus of elasticity test results verified that the 28-day compressive strength and modulus of elasticity meets or exceeds the design requirements. The results also verified that both compressive strength and modulus of elasticity increase with age.

Structural Analysis of the Composite Wall for Handling Loads

It is generally assumed that the most severe handling case is when the pipe is resting on line bearing (0º bedding angle) and subjected to 50% of the pipe weight as an impact force. The main objective of the analysis is to study the structural behavior of the liner, from a cracking point of view, during pipe handling and installation. Structural analysis was conducted on many reinforcement schemes for the concrete core of the composite liner using case 18 from Roark's Formulas for Stress and Strain (IV Ed.), for forces and ring deflections due to pipe weight on line bearing. The trial analysis was based on designs with a minimum cylinder thickness of 0.3125 in., 9 in., 10 in., and 12 in. concrete core, and one or two reinforcing cages in the concrete core.

Based on the preliminary analysis the option with the 9 inch thick concrete core was eliminated. Structural design was conducted on two final designs for 12 in. core and 10 in. core, respectively.
The final analysis assumed the following design parameters:

- Inside diameter = 144 in.
- Concrete core thickness = 10 in. or 12 in.
- Steel cylinder I.D = 164 in. or 168 in.
- Cylinder thickness = 0.3125 in.
- Concrete compressive strength = 5500 psi
- Inner reinforcing cage area = 0.84 in^2/Lf. deformed bars
- Outer cage = 0.60 in^2/Lf. Smooth bars
- Concrete cover for inner and outer cage = 1.5 in.

- One-half inch diameter, 8 inch long for the 12 in. core and 6 in. long for the 10 in. core, Nelson Studs spaced at 24 in. x 24 in. except in the invert area the spacing is 12 in. x 24 in.

The analysis calculated maximum horizontal and vertical deflections based on uncracked section and cracked section, using a 1.5 impact factor. Maximum stresses due to moment and thrust at the pipe invert and springline were also calculated.

Pipe Deflections

Summary of pipe deflections with 1.5 impact factor and line bearing conditions is**:**

Core Thick.	Maximum Deflection, in.			
	Uncracked		Cracked	
	Horizontal.	Vertical	Horizontal	Vertical
10 in.	0.041	0.018	0.061	0.027
12 in.	0.031	0.014	0.034	0.015

The results indicated that the maximum deflections under extreme conditions are very small and that there is no need for any stulling. The deflections without the 50 % impact will be only 67% of the values listed above. The reinforcement scheme inside the steel cylinder is shown in Figure 3.

Maximum Ring Bending Stresses

Using the transformed section properties of the liner and the moment and thrust coefficients derived for the Roark's Case 18, the maximum stresses are:

Core Thickness	Max. Tensile Stress at Invert, psi	Max. Tensile Stress at Springline, psi
10 in.	529	150
12 in.	460	133

The maximum tensile stresses are less then the allowable 556 psi ($7.5 \sqrt{f'_c}$) for 5500 psi concrete. However if cracking does occur, the maximum crack width using the Gergely-Lutz formula will not exceed 0.0061 in. and 0.0065 in. at the invert for the 12 in. core and 10 in. core, respectively.

Check for Shear in Pipe Core

The shear load was determined using the Roark shear coefficients. The shear analysis was based on AASHTO LRFD Bridge Design Specification Sec. 5.8, 1994. Summary of results are:

Figure 3. Reinforcing Cages and the Nelson Studs inside the Cylinder

Core Thickness	Max. shear, Kip/ft	Nominal Shear Resistance, Kip/ft	Margin of Safety
10 in.	4.1	5.94	1.44
12 in	4.9	7.1	1.45

The results showed that there is no need for shear reinforcement.

Shear Connection Between Steel Cylinder and Core Concrete

One-half inch diameter studs at a spacing of 24 in. x 24 in were used as shear connections throughout the core. The shear resistance of the studs was checked in conformance with AASHTO LRFD Bridge Design Specification Sec. 6.10.7.4, 1994. The analysis showed that in order for the studs to be effective at the location of maximum shear in the invert area, the spacing should be 24 in. x 12 in.

Radial Tension Check

A radial tension check was conducted in conformance with the Concrete Pipe Technology Handbook, ACPA, section 7.5.3.1, 1993. Summary of results are:

Core Thick.	Radial Tension Stress, psi	Radial Tension Strength, psi	Margin of Safety
10 in.	20.8	64.1	3.08
12 in.	20.4	64.1	3.14

The analysis showed that the margin of safety of for radial tension is satisfactory.

Testing For Design Verification

Since this was the first application of such a tunnel liner concept, tests were

Figure 4. 3-Edge Bearing Load-Deflection Test

conducted on two test pipe sections manufactured in conformance with the final proposed reinforcement designs for the 10 in. and 12 in. cores, respectively. The objective of the test was to verify the designs and study the structural behavior, from a deflection and cracking point of view, of the two liners during pipe handling and installation. The lengths of the two test pipe sections were 8 ft.

To simulate as close as possible to line bearing condition, the test pipe was subjected to the 3-edge bearing test in conformance with ASTM C497. Once the test pipe is set in the test machine as shown in Figure 4, the pipe will be resting at two points, 12 inches apart and subjected to stresses resulting from its own weight. Then, any load applied over the pipe, through a beam and hydraulic rams will be additional to its own weight.

Test Instrumentation

Four deflectometers were installed inside each 8-ft long test pipe section. Two of the deflectometers were placed vertically to measure the vertical deflection; the other two were placed horizontally to measure the horizontal deflection. One of each vertical and horizontal deflectometers was placed about 30 inches from each end of the steel cylinder. Two strain gages were attached to the interior concrete surface at the top and bottom and to the exterior steel surface at each springline. One strain gage at each location was placed 24 inches from each end of the steel cylinder. All strain gages were oriented in the circumferential direction. A continuous recording data logger was connected to the four deflectometers, strain gages and hydraulic rams of the 3-edge bearing load through a pressure transducer.

Test Procedure

The first load on the two test pipe was the 50% impact load, which is 3.3 Kip/ft and 2.8 Kip/ft for the 12 in. core and 10 in. cores, respectively. The load was kept constant and test pipe sections were inspected for cracking. The load was increased to higher levels and test pipe was inspected for cracking. The maximum loads causing the 0.01 in. crack (a selected crack width that is used for testing comparisons) were decreased slowly to zero loads.

Test Results and Observations

Summary of deflections and loads are:

Core Thick.	Deflections at 50% Impact Load, in.		Deflections at First Crack Load, in.		Deflections at 0.01" Crack Load, in.	
	Horizontal	Vertical	Horizontal	Vertical	Horizontal	Vertical
10 in.	0.024	0.02	0.081	0.09	0.42	0.47
12 in.	0.01	0.007	0.0 44	0.040	0.24	0.27

The load deflection curve for test pipe with 12 in. is given in Figure 5. The load deflection curve for the 10 in. core is not shown but the results are included. The load versus time is not shown; however, the breaks observed in the load/deflection curves are where the load was kept constant for approximately 15 minutes to allow for inspection of the concrete for cracking. The strain readings were not dependable.

The loads at which the first visible crack was observed for the 10 in core and the 12 in. core were 5.4 kip/ft and 7.1 kip/ft, respectively. These loads are equivalent to approximately 100% impact loads. The measured deflections at the 50% impact loads were less than the calculated deflections.

The loads which caused the 0.01 in. crack were approximately 18 kip/ft and 16 kip/ft for the 12 in. and 10 in. core liners, respectively which are significantly higher than the anticipated loads. The vertical deflection of the test pipe sections with the 12 in. and 10 in. cores were approximately 0.3 in. and 0.5 in., and the deflections after unloading were 0.08 in and 0.15 in. This indicates good elastic recovery.

The test results verified that the structural analyses of the pipe were conservative and the pipe acts as a composite wall and can withstand all anticipated handling and installations forces.

Ad**vantages of the Composite Wall Design**

The composite steel-and-concrete wall designs for the high external load applications offer the following advantages and cost savings:

- More economical to produce than steel pipe (For example, 1200 ft. external head design for 144 in. steel liner may require 2.5 inch thick steel).
- Rigid wall is easier to handle and install than steel pipe.
- Requires cellular grout only instead of conventional grout for steel pipe.

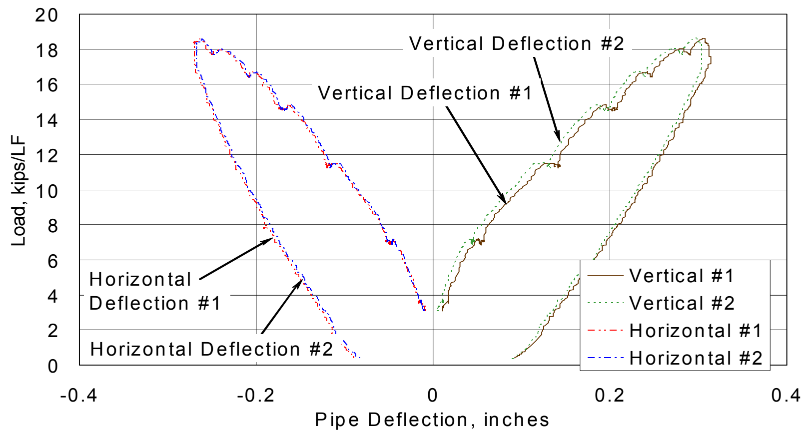

Figure 5. Load-Deflection Curve for 144-inch Tunnel Liner with 12-inch Concrete Core

Conclusions

- The design of the Arrowhead tunnel liners is unique due to its large 144-inch diameter and exposure to 1200 feet of external hydrostatic pressure; there are no known similar applications.
- The design concept of the composite steel and concrete liner is based on having the concrete core resisting over 60% of the external pressure and the steel cylinder resisting the remainder of the external pressure and 100% of the internal pressure.
- The main challenges are the design and testing of the reinforcement scheme of the concrete core to ensure that the structural integrity of the liner is not compromised during pipe handling and installation, and also the selection and production of high-quality concrete to ensure that there would be no overstress in the steel cylinder for the long term condition.
- The composite wall design for large diameter tunnel liners exposed to high external pressure offers many advantages and cost savings.

References

American Association of State Highway and Transportation Officials (1994). "AASHTO LRFD Bridges Design Specification", *Sec. 5.5 & Sec 6.10.7.4.*

ACI Committee 209 (1982). Report No. ACI209R-82. "Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures", *SP-76-10: 193-300,* Detroit, American Concrete Institute.

Gergely, P. & Lutz, L. A. (1968). "In Causes, Mechanism, and Control of Cracking in Concrete". *SP-20:87-117*, Detroit, American Concrete Institute.

Roark, R.J., "Formulas for stress and Strain", *IV Ed., p.176, case 18*, McGraw-Hill.