Design of Prestressed Concrete Pressure Pipe

by Harold V. Swanson*

INTRODUCTION

In 1942 the first prestressed concrete cylinder pipe was produced commercially. Production of prestressed concrete embedded cylinder pipe began some ten years later, fulfilling a demand for higher pressures and larger diameters. In the past two decades millions of feet of these two types of prestressed pipe have been installed, both in this country and abroad in sizes from 12 in. to 120 in. diameter. The Standards for these pipe are specified in AWWA C301.

During this same period studies were made on the economy of producing a prestressed pipe with an all-concrete core; that is, without a steel cylinder. By 1952 a prototype had been developed and three successful projects, totaling 90,000 feet, had been installed in Latin America. From 1952 to the present, developmental efforts were directed toward improved manufacturing and design techniques to achieve maximum economy consistent with conservative design. In recent years several contracts have been obtained using this improved product.

In the design of any structure the engineer has as his principal objective the use of materials to their utmost capabilities, consistent with safety, to achieve maximum economic advantage. He can only do this through a complete understanding of the laws of mechanics which govern these materials. The application of the principle of prestressing in the design of concrete structures achieves this desired objective; a rigorous approach to design and a thorough reconciliation of design to performance are necessary.

GENERAL CHARACTERISTICS

Prestressed concrete pressure pipe (Figs. 1 and 2) is capable of operating over a wide range of pressure ratings; typical ratings for minimum wall designs are shown in Table 1. By various combinations of wall thickness and wire area, it is possible to utilize prestressed concrete pressure pipe in virtually all operating conditions encountered in water works practice. It is significant to note that we have tested 16 in. pipe, with a 1⅞ in. core, to 900 psi without leakage. Prestressed concrete is an especially effective solution to the physical and economic problems of designing a high pressure water pipe. Concrete is relatively weak in tension, but strong in compression. Prestressing takes advantage of the

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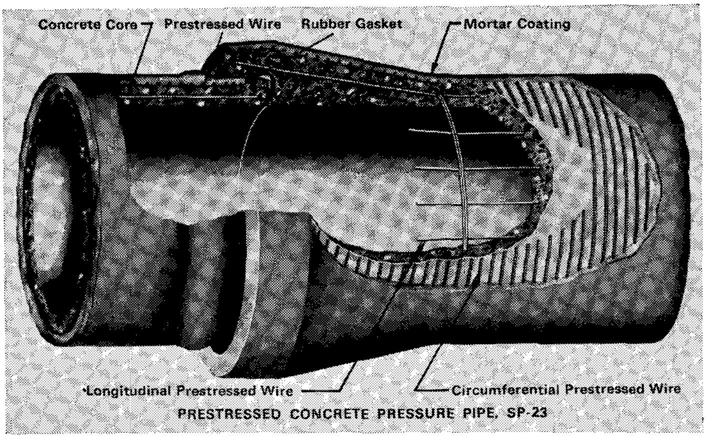

Fig. 1—Prestressed Concrete Pressure Pipe with Rubber and Concrete Joint

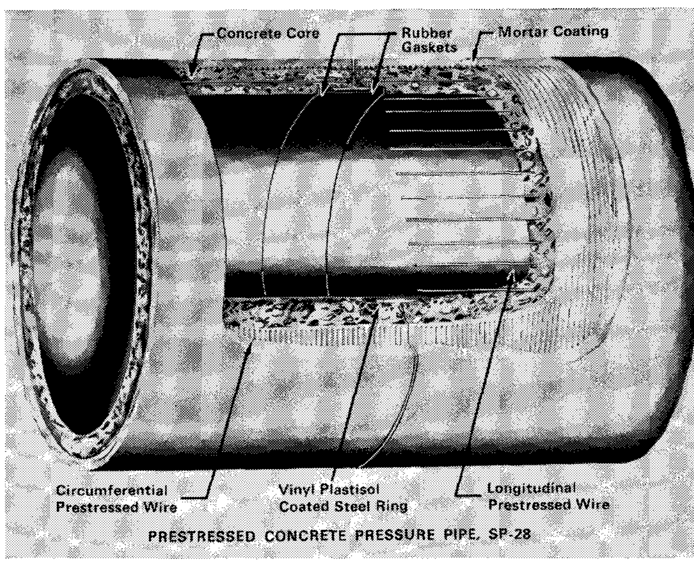

Fig. 2—Prestressed Concrete Pressure Pipe with Double Rubber and Concrete Bell Joint

latter characteristic by placing the core in compression. This has the effect of enormously increasing the tensile stress range of the concrete in resisting internal pressure, thereby extending the capabilities of the pipe far beyond the limits of conventionally reinforced non-prestressed pipe. In order that the pipe may withstand these high pressures, the concrete must be dense and impervious. This high quality concrete, essential for good performance, results in the additional advantages of good structural behavior, minimum plastic flow and sustained hydraulic carrying capacity, combined with maximum resistance to corrosion.

This type of pipe is prestressed longitudinally as well as circumferentially. Longitudinal prestressing provides the beam strength neces-

Table 1

Typical Ratings With Minimum Wall

DIAMETER IN.	CORE THICKNESS IN.	ALLOWABLE WORKING PRESSURE PSI
16	1⅞	350
18	1⅞	350
20	1⅞	325
24	1⅞	320
30	1⅞	250
36	2¼	250
42	2⅝	250
48	3	250
54	3⅜	250
60	3¾	240
66	4⅛	240
72	4½	230

Maximum rating for minimum wall prestressed concrete pressure pipe under 5 feet of cover. Higher pressures are permissible with thicker cores.

sary in normal installations and overcomes secondary transient and permanent stresses resulting from the circumferential prestressing. It also reduces the principal shearing stress which occurs across the joint. In addition, circumferential plastic flow is reduced because the core is in a state of tri-axial stress.

The medium used to provide the prestressing in the concrete is high tensile steel wire. As in the case of the concrete, the capability of steel is utilized to its best advantage in this structure.

The completed prestressed concrete core has a covering of mortar on its exterior surface. The function of the mortar coating is to protect the prestressing wire from corrosion. This protection is derived from the alkaline environment which the rich mortor provides at the concrete-to-steel interface, enobling the metal by the formation of a passivating film. It is significant to note that the mortar, combined with a spray of neat cement slurry directly on the wire, does not protect the steel by keeping moisture away from the wire. Rather, the mortar-slurry system precludes the replenishment of free oxygen which, in combination with chloride or sulfide ions, is necessary to disrupt the passivating film. Following the spray of neat cement slurry, the mortar is impacted on the prestressed core at high velocity. The resulting bond between the core and the coating, and the mechanical interlocking action between the prestressing wire and the coating allow the latter to withstand large strains prior to the formation of visible cracks. The magnitude of those strains is most strikingly demonstrated by the ability to test coated pipe well in excess of its rated working pressure before the occurence of visible cracks (0.002″).

SERVICE LOAD DESIGN

Design of the pipe to meet service conditions considers the combined effect of internal pressure and external load. The stresses caused by these service conditions are added algebraically to the induced core compression. The net resultant stresses determine safe combinations of internal pressure and external load for any particular design.

Allowable operating pressures are related to P_{co}, the hydrostatic pressure, in pounds per square inch, which results in zero stress in the core concrete at the point of maximum tensile stress change under combined load. Depending on specific design requirements and anticipated service conditions, a relationship is established between the working pressure, P_w, and the combined load zero compression pressure, P_{co}. As a general guide, for normal service conditions the ratio P_{co}/P_w should be 1.00; however, under unusual conditions it may be modified.

P_{co} is expressed as:

$$P_{co} = \frac{t_c}{R_i}\left(f_{cr} - \Delta f_{cw}\right)\left(1 + n_r \frac{A_s}{A_c}\right)$$

where:

t_c = Core thickness, in.

R_i = Internal radius of pipe, in.

f_{cr} = Resultant core compression, psi.

Δf_{cw} = Stress change caused by external loading, psi.

A_s = Area of prestressing wire, in²/lin. ft.

A_c = Area of core, in²/lin. ft.

$n_r = \frac{E_s}{E_{cr}}$ = Final modular ratio

$= \frac{25.5 \times 10^6}{5.10 \times 10^6} = 5$

E_s = Modulus of elasticity of prestressed wire

E_{cr} = Final modulus of elasticity of concrete

RESULTANT CORE COMPRESSION

The resultant core compression f_{cr} used in the above formula, apart from elastic losses, must take into account the loss in compression which occurs between the time the pipe is initially prestressed and when it is finally placed in service. This loss involves relaxation of the high tensile wire, embedment of the wire into the concrete core and the inelastic creep of the concrete, occurring as a result of sustained stress.

The final core compression, f_{cr}, is expressed by the formula:

$$f_{cr} = \left[\frac{A_s f_{sg}(1-R_1-R_2)}{A_c + n_i A_s}\right] \times \left[\frac{A_c + n_r A_s}{A_c + n_r A_s(1 + C_r)}\right]$$

where

R_1 = Wire relaxation loss factor = 0.05

R_2 = Wire embedment loss factor = 0.05

$n_i = \frac{E_s}{E_{ci}}$ = Initial modular ratio

$= \frac{25.5 \times 10^6}{4.25 \times 10^6} = 6$

E_{ci} = Initial modulus of elasticity of concrete

C_r = Concrete creep factor (ratio of inelastic strain to elastic strain) = 1.5

High tensile wire at normal temperature is subject to relaxation of stress when it is maintained under tension at a constant length. Wire embedment loss is related to the surface strength of the concrete and the magnitude of the concentrated radial load of the prestressing wire. When the wire is wrapped on the surface of the core, it will embed itself due to local plastic flow of the concrete. This will continue until the radial load of the wire is equal to the supporting capacity of the concrete. The embedment loss is taken as 5% of the initial gross wrapping stress.

It is well known that concrete does not exhibit a strict linear relationship between stress and strain; however, the deviation is so slight within the usual working stress range that it has been universally assumed that the modulus does not vary with the applied load. As the concrete ages, however, the modulus increases. This characteristic of concrete has been confirmed by tests and is recognized in the design procedure.

Creep of concrete is an inelastic time-dependent deformation resulting from the presence of stress (Fig. 3). It is expressed as a ratio of the inelastic strain to the elastic strain of the concrete. Creep proceeds at a rapid rate initially and asymptotically approaches a maximum after two or three years when the concrete is subjected to a constant com-

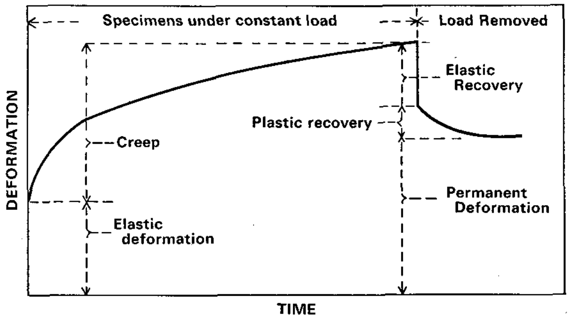

Fig. 3—Qualitative Representation of Creep in Concrete

pressive stress. The limiting value of creep strain, e_c, is denoted by:

$$e_c = \frac{C_r f_{cr}}{E_{cr}}$$

This value is the gross strain which would occur in an unrestrained specimen. However, inelastic creep strains occurring in prestressed concrete pipe are accompanied by elastic reactions in the prestressing wire and concrete. Therefore, the loss in stress in the core, Δf_c, due to creep strain is determined by:

$$\Delta f_c = \frac{n_r A_s C_r f_{cr}}{A_c + n_r A_s}$$

The time-dependent nature of creep strain has been evaluated using the general expression of Caquot[1],

$$X = 1 - 10^{-K_1 D^{K_2}}$$

where

X = Proportion of final creep strain

D = Time in days after prestressing

K_1 and K_2 = Constants, dependent on concrete quality

Analysis of creep strains of specimens subjected to compressive stress at our Research Center over a period of several years allowed us to evaluate the constants K_1 and K_2. The resulting expression is:

$$X = 1 - 10^{-0.1055 D^{0.457}}$$

The expression has significance when it is known that the pipe will be placed in service before full losses have occurred. When the pipe is pressurized, compression of the core is relieved and creep is arrested.

Shrinkage of the concrete is not treated separately in the consideration of inelastic losses. Some shrinkage takes place prior to the prestressing of the core, but this has no effect on the final compression. Shrinkage occurring after prestressing is included in the value of the creep factor due to the method of determining creep; i.e. hydrostatic tests of aged pipe, atmospherically stored. This treatment of shrinkage is conservative because: first, it is doubtful that any residual shrinkage will be sustained by a pipe in service due to its moist external and saturated internal environment; and, second, pipe which has sustained shrinkage, due to atmospheric storage, will exhibit a reversal of shrinkage when subsequently exposed to service conditions.

STRESS CHANGES FROM EXTERNAL LOAD

Stress change due to the effect of external loads is determined at the points of maximum bending moment, the invert and the springline. The primary loading is due to the weight of earth covering the pipe. In addition, both the weight of the pipe, and the weight of water in the pipe are considered in design. These loads are assumed to be distributed as shown in Figs. 4, 5 and 6.[2] Moment and thrust coefficients are shown in Tables 2 and 3. Based on this distribution and the specific bedding class, moments and thrusts are determined by elastic analysis. The resulting stress change in the core, Δf_{cw}, is then determined by:

$$\Delta f_{cw} = \frac{M}{2t_w^2} - \frac{T}{12t_w}$$

where

M = Summation of the bending moments at the invert, or springline, whichever is greater, in-lbs/ft.

T = Summation of the thrust at the invert or springline, lbs/ft.

t_w = Wall thickness, in.

CIRCUMFERENTIAL DESIGN SUMMARY

Summarizing these expressions, a straightforward design procedure becomes apparent:

1. Assuming an area of prestressing wire, A_s, determine resultant core compression, f_{cr}.

$$f_{cr} = \left[\frac{A_s f_{sg}(1\text{-}R_1\text{-}R_2)}{A_c + n_i A_s}\right]\left[\frac{A_c + n_r A_s}{A_c + n_r A_s(1 + C_r)}\right]$$

2. With a known earth load and weight of pipe filled with water, de-

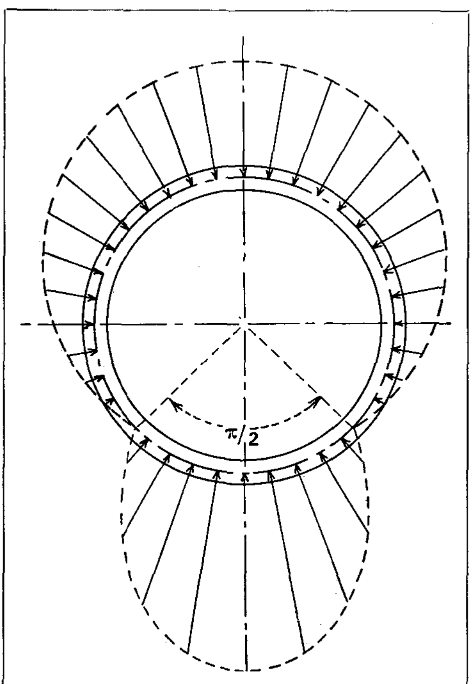

Fig. 4—With a Central Angle of $\pi/2$, Earth Pressures and Reactions Exhibit the Characteristic Bulb Shapes

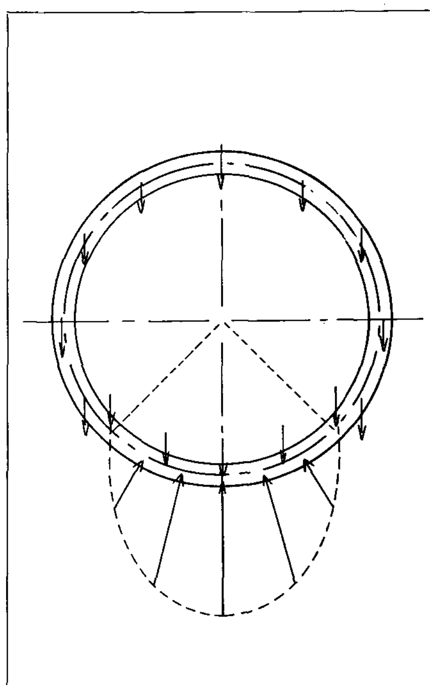

Fig. 5—Assumed Distribution of Dead Load Due to Weight of Pipe

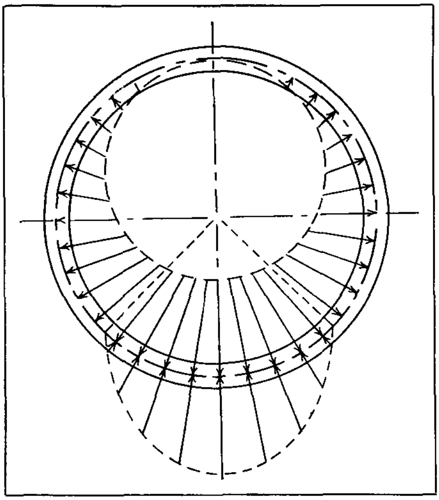

Fig. 6—Assumed Pressure Distribution of Water Load

Table 2

Moment and Thrust at Invert

	FIRST CLASS BEDDING Bedding Angle $\frac{\pi}{2}$		ORDINARY BEDDING Bedding Angle $\frac{\pi}{4}$	
Weight (of)	Moment In-Lbs/LF	Thrust Lbs/LF	Moment In-Lbs/LF	Thrust Lbs/LF
Backfill	$0.126\,W_eR_m$	$0.324\,W_e$	$0.160\,W_eR_m$	$0.36\,W_e$
Pipe	$0.122\,W_pR_m$	$0.207\,W_p$	$0.175\,W_pR_m$	$0.15\,W_p$
Water	$0.122\,W_wR_m$	$-0.272W_w$	$0.175\,W_wR_m$	$-0.33\,W_w$

Table 3

Moment and Thrust at Springline

	FIRST CLASS BEDDING Bedding Angle $\frac{\pi}{2}$		ORDINARY BEDDING Bedding Angle $\frac{\pi}{4}$	
Weight (of)	Moment In-Lbs/LF	Thrust Lbs/LF	Moment In-Lbs/LF	Thrust Lbs/LF
Backfill	$0.089\,W_eR_m$	$0.539\,W_e$	$0.082\,W_eR_m$	$0.60\,W_e$
Pipe	$0.088\,W_pR_m$	$0.297\,W_p$	$0.095\,W_pR_m$	$0.31\,W_p$
Water	$0.088\,W_wR_m$	$-0.062\,W_w$	$0.095\,W_wR_m$	$-0.05\,W_w$

termine bending moment and thrust at the invert and at the springline.

3. Compute maximum tensile stress change due to bending and thrust.

$$\Delta f_{cw} = \frac{M}{2t_w^2} - \frac{T}{12t_w}$$

4. Determine the internal pressure, P_{co}, which results in zero core stress, after compensating for stresses caused by earth load, pipe and water weight:

$$P_{co} = \frac{t_c}{R_i}\left(f_{cr} - \Delta f_{cw}\right)\left(1 + n_r\frac{A_s}{A_c}\right)$$

5. Determine allowable working pressure:

$$P_w = P_{co}$$

LONGITUDINAL DESIGN

The amount of longitudinal prestress depends on two factors; resistance to beam loading, and resistance to transient and permanent bending stresses developed during the process of wrapping.

For large diameter pipe, beam stresses are insignificant. For the smaller pipe, however, beam loading can be significant. Therefore, all pipe are designed to withstand a simple beam load of 500 pounds per square foot on the projected area, plus the weight of the pipe full of water, without cracking. The assumption that the pipe acts as a simply supported beam is a conservative one, since there will be some support from the sub-grade.

Thus, required resultant longitudinal compression, f_{cr}, due to beam action is determined by:

$$f'_{cr} \geqq \frac{(w_e + w_p + w_w)\,L^2D_o}{8\,[.0982(D_o^4\text{-}D_i^4)]} - 4\sqrt{f'_c}$$

where

w_e = weight of uniformly distributed earth load, lbs./in.

w_p = weight of pipe, lbs./in.

w_w = weight of water in pipe, lbs./in.

L = pipe span, in.

D_o = outside diameter of pipe (including coating), in.

D_i = internal pipe diameter, in.

f'_c = 28-day strength of concrete, psi.

$4\sqrt{f'_c}$ = allowable tension with margin against cracking, psi.

PREVENTION OF CRACKING DUE TO SECONDARY BENDING STRESSES

Secondary transient stresses due to wrapping result from the elastic deformation of the core as the wire is being placed. The theoretical value of the transient stress in the barrel is 28% of the initial circumferential core compression. Longitudinal prestress is provided to prevent cracking in the barrel at points distant from the ends of the pipe. Thus

$$f'_{ci} \geqq .28 f_{ci} - 500$$

where

f'_{ci} = Initial longitudinal prestress, psi

f_{ci} = Initial circumferential prestress, psi

500 = Allowable flexural stress for no cracking, psi.

Other secondary stresses due to wrapping occur at the ends of a pipe with a spigot end due to the impracticability of wrapping completely to the spigot end. While longitudinal prestress will have a tendency to overcome the spigot bending stress, additional spigot cage reinforcement is needed to distribute the tensile strains and to control the effect of cracking. The factors which determine the amount of this non-prestressed reinforcement depend upon the rate of development of longitudinal prestress, the use of positive end anchors, the pipe wall thickness, the distance of the last wrap from the spigot end and the spacing of the wire in the spigot cage. Combining all these factors into a semi-empirical expression, the minimum longitudinal prestress is expressed as

$$f'_{ci} \geqq k_i f_{ci} - 50{,}000\, p'' - 500$$

Where p'' is the ratio of the longitudinal spigot cage area to the cross-sectional area of the core normal to the pipe axis.

The factor k_i, Fig. 7, is a bending stress coefficient whose value is dependent upon the dimensions of the pipe, B, and the distance, b, in inches, from the end of the spigot to the first wrap of prestressing wire. The factor, B, is expressed as

$$B = \left[\frac{1.71}{t_c R_{mc}}\right]^{0.5}$$

where

t_c = core thickness, in.

R_{mc} = mean radius of core, in.

LABORATORY TESTS

Confirmation of the foregoing design equations and material physical characteristics has been based on various tests performed at our Wharton Research Center.

HIGH TENSILE STEEL

Relaxation of high tensile wire under tension is a result of the readjustment of the grain structure of the stressed steel. The amount depends upon the applied tension and the type of wire. Relaxation approaches its limit in about one month

Table 4
Relaxation of Steel

Diameter In.	Ultimate Stress psi	Initial Stress		Duration of Test Hours	Relaxation %
		psi	% of Ult.		No Initial Overstress
0.20	200,000	134,000	67	380	4.5
0.276	200,000	134,000	67	380	4.6

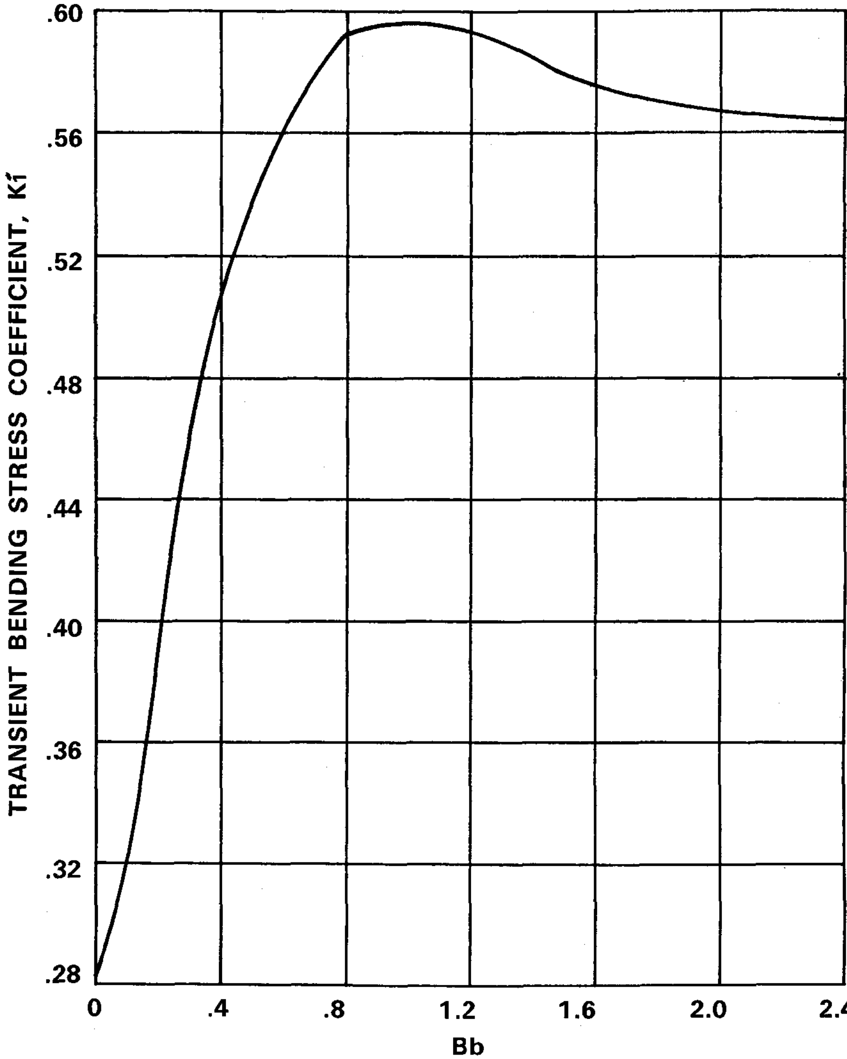

Fig. 7—Bending Stress Coefficient

with 75% occurring within the first day and 50% occurring within one hour. Cold working and application of a slight overstress reduces the amount of relaxation (See Table 4).

The modulus of elasticity of the hard-drawn steel spring wire, ASTM A227, has been established on the basis of stress-strain curves in which the wire is proof-loaded to a level in excess of the gross wrapping stress. Proof loading, which is accomplished in the prestressing machine as the wire passes over the stress inducing sheave, has the effect of raising the proportional elastic limit and achieving a linear relationship between stress and strain. From the curves of the types shown in Figs. 8 and 9, the average modulus of elasticity of the wire is found to be 25,500,000 psi.

CONCRETE CREEP FACTOR

The creep factor used in design of prestressed concrete pressure pipe has been evaluated on the basis of hydrostatic tests on pipe which are of sufficient age to have undergone full losses. These tests have been conducted on pipe which has been initially cracked under hydrostatic pressure. The zero compression pressure, P_o, is recorded when these cracks reopen under test. Fig. 10 is a plot which shows test P_o values in relation to theoretical P_o values computed on the basis of a creep coefficient of 1.5. The radius-core thickness ratio is included in the plot in order to make the graph apply to the variety of pipe sizes which were tested. The maximum age of pipe tested was 7½ years. The graph indicates the conservatism of the creep coefficient chosen for design. Further conservatism results from the method of determination. This has been done on pipe aged by atmospheric storage, and, therefore, subject to drying shrinkage and higher sustained stress levels than pipe in service.

HYDROSTATIC TESTS

The analytical expressions which describe hydrostatic performance have been consistently confirmed by testing of 16 in. through 84 in. pipe. The base line to which all final designs are related is P_o, the pressure which results in complete dissipation of the induced final core compression with no external load. The expression for P_o is:

$$\frac{P_o R_i}{t_c} = f_{cr}\left(1 + \frac{A_s}{A_c} n_r\right)$$

Fig. 11 compares test results with the theoretical design expression for P_o. These tests were conducted on pipe 1½ to 3 years old which had experienced virtually full inelastic losses.

THREE-EDGE BEARING TESTS

A convenient test for ring strength

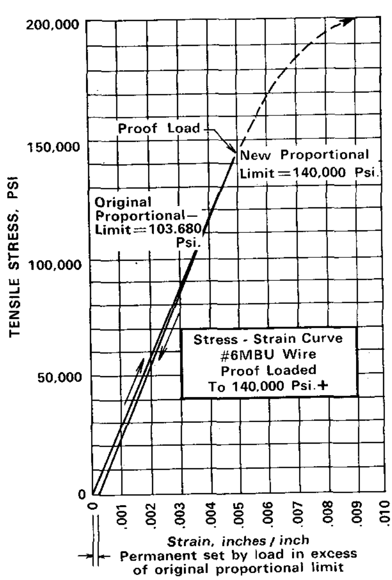

Fig. 8—Stress-Strain Curve of #6MBU Wire Proof Loaded to 140,000 psi Plus

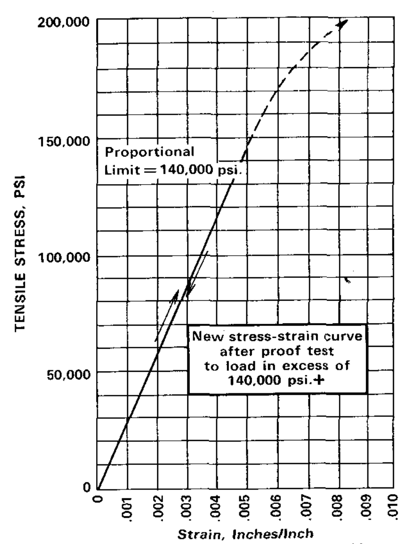

Fig. 9—Stress-Strain Curve of Wire after Proof Test to Load in Excess of 140,000 psi Plus

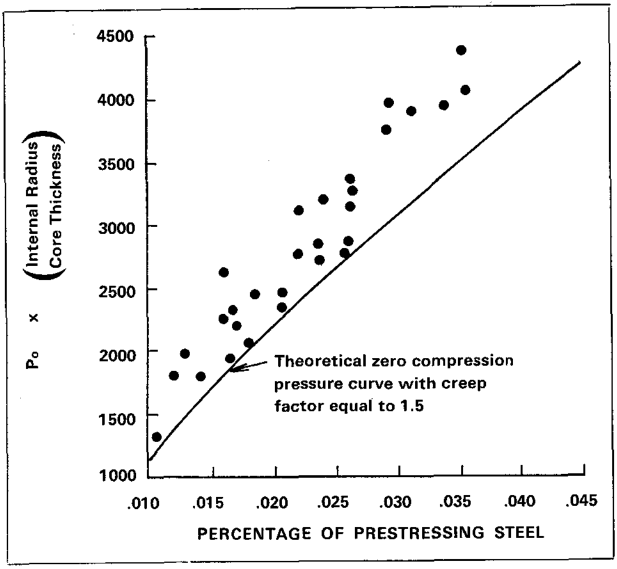

Fig. 10—Plot of Test P_o in Relation to Theoretical P_o for Creep Coefficient Equal to 1.5

of pipe is the three-edge bearing test, ASTM C-76. For this test, the pipe is placed on two wooden supporting strips and loaded by a hydraulic testing machine through a wooden strip along the crown. The ability of pipe to resist such external loads is derived from the strength of the section as a prestressed closed circular ring. When designing pipe to resist such loading, it is assumed that the stress changes may be computed elastically according to the principles of mechanics. Table 5 indicates the factor of safety that the stress condition of zero and 7.5 times the square root of the concrete compressive strength (allowable core flexural strength) have against cracking by external loading.

BEAM TESTS

The strength of prestressed concrete pressure pipe against beam loading is dependent upon the amount of longitudinal compression induced into the core and on the modulus of rupture of the concrete in the core. Table 6 shows the maximum bending moments based on

Table 5—Factor of Safety Against Cracking by External Loading

PIPE DATA				TEST 3-EB LOAD	DESIGN 3-EB LOAD		SAFETY FACTORS	
Dia. In.	Core In.	Coating In.	Circum. PS Wire In²/LF	At .002" Crack Lbs/LF	At Zero Comp. Lbs/LF	At 7.5 $\sqrt{f'_c}$ Tension Lbs/LF	.002" Ck. Load / Zero Comp. Load	.002" Ck. Load / 7.5 $\sqrt{f'_c}$ Tens. Load
16	1 ⅞	⅞	.522	21,700	11,260	14,210	1.93	1.53
30	2	⅞	.238	10,460	3,480	5,320	3.01	1.97
30	2	⅞	.238	11,030	3,480	5,320	3.18	2.07
30	2	⅞	.638	19,070	7,780	9,620	2.45	1.98
30	2	⅞	.638	23,310	7,780	9,620	3.00	2.42
30	2	⅞	.667	20,000	8,000	9,840	2.50	2.03
30¼	1 ⅞	⅞	.740	18,300	8,300	9,980	2.20	1.83

Table 6—Test and Design Bending Moments

Dia. in.	Core Thick. in.	Coating Thick. in.	No. and Size of Longit.	Test: Max. Mom. at 1st Crack in.-lbs.	Test: Max. Mom. at Failure in.-lbs.	Design: *Allow. Resisting Mom. in.-lbs.	Design: At 1st Crack S.F.	Design: At Failure S.F.
16	1 ⅞	⅞	12- #6	0.738×10^6	0.862×10^6	0.477×10^6	1.55	1.81
16	1 ⅞	⅞	12- #6	0.841×10^6	0.841×10^6	0.477×10^6	1.76	1.76
24	1 ⅞	⅞	20- #6	1.651×10^6	1.912×10^6	1.090×10^6	1.52	1.76
30	1 ⅞	⅞	24- #6	2.626×10^6	2.876×10^6	1.600×10^6	1.64	1.80
30	1 ⅞	⅞	24- #6	2.576×10^6	3.176×10^6	1.600×10^6	1.61	1.98

$f'_{sg} = 130{,}000$ psi for #6 MBU
*Allowable Tension in Core $= 4\sqrt{f'_c} = 310$ psi
Allowable Strain in Coating $= 30 \times 10^{-5}$

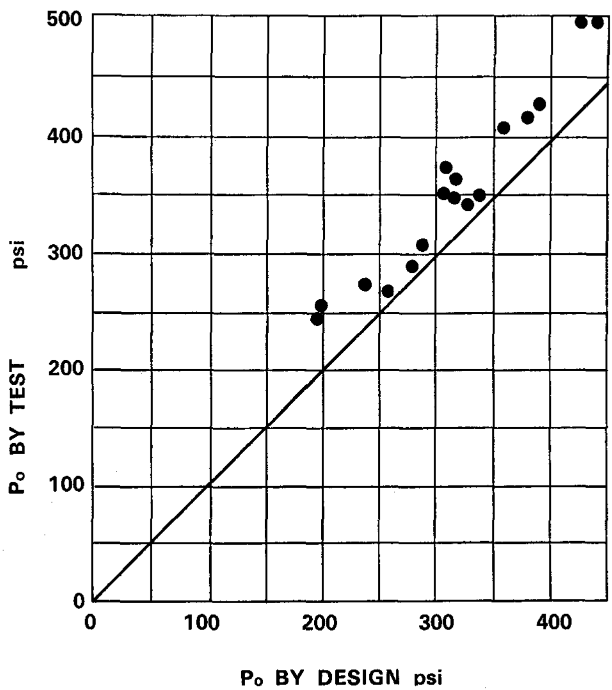

Fig. 11—Comparison of Design P_o and Test P_o

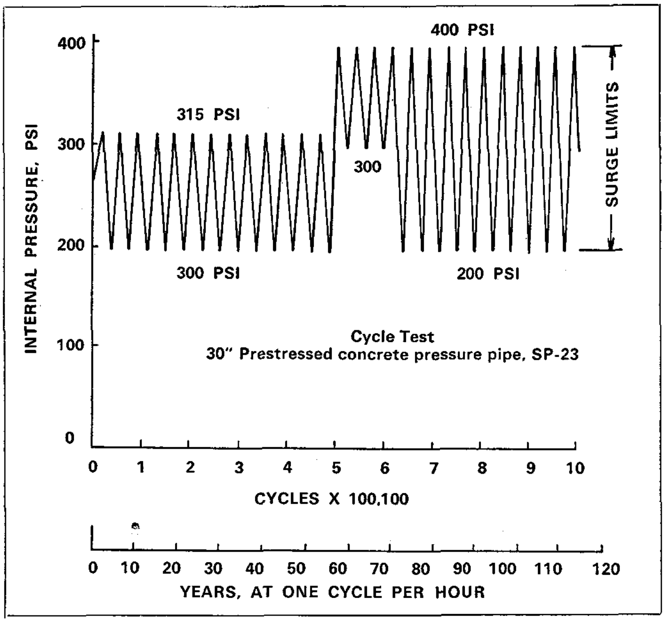

Fig. 12—Cycling Test History for 30 in. Pipe (Core thickness = 2 in., Coating thickness = 7/8 in.)

test results, together with corresponding design resisting moments.

Although the average factor of safety as related to tested strength and allowable design strength is in the order of 1.5, it should be pointed out that a further margin against cracking is obtained by the conservative assumptions used when investigating any specific design subjected to field loading. These assumptions involve considering the pipe as a simple beam supported on knife-edge bearings with a span equal to its full length. However, it is highly unlikely that there will be a complete lack of support along the bottom. It is difficult to state quantitatively what this increase in the factor of safety would be since the characteristics of the supporting soil cannot be determined in each case. Considering, however, a 16 in. pipe supported on a elastic foundation of ordinary soil, a theoretical analysis yields an additional factor of safety of 4 over that condition where no elastic support is assumed.

SURGE TESTS

One of the most critical appraisals of a pipe's performance under actual operating conditions is obtained by a hydrostatic cycling test exceeding its design pressure. The number of cycles to produce deleterious effect is an indication of the relative safety factor of various types of concrete pipe. The cycling history of a 30 in. prestressed concrete pressure pipe designed for an operating pressure of 192 psi is shown in Fig. 12. The pipe was surged 500,000 times between the limits of 200 psi and 315 psi. Then 100,000 cycles between 300 psi and 400 psi were applied followed by an additional 400,000 cycles between 200 psi and 400 psi.

The test was discontinued after one million cycles and 27 months without producing harmful results in the core or in the coating.

CONCLUSION

Prestressed concrete pressure pipe can be engineered for specific installations, allowing the engineer to satisfy his requirements with the optimum efficiency and economy. Tests of completed pipe show a remarkable correlation between theory and performance.

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