

# Design of Circular Concrete Pipe Reinforced With Welded Deformed Wire Fabric

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The structural behavior of circular concrete pipe reinforced with welded deformed wire fabric has been investigated relative to the performance requirements of ASTM C 76. Three-edge bearing tests, as defined in ASTM C 497, were carried out on 70 full-scale pipe specimens. Theoretical analyses of cracking, ultimate flexural and ultimate diagonal tension behavior are developed. Equations are presented for 0.01-in. crack strength and ultimate strength. Where necessary, test results were used to determine semi-empirical constants in these equations. A complete design procedure is suggested for C 76 pipe with welded deformed wire fabric reinforcing.

The results of the investigations indicate that welded deformed wire fabric reinforcing controls crack width more effectively than the various non-deformed reinforcements now in common use for concrete pipe. Consequently, for sizes and strength classes where steel area requirements are governed by 0.01-in. crack strength criteria, C 76 performance requirements may be achieved with significant reductions in quantity of reinforcing steel with the use of welded deformed wire fabric reinforcing. In general, this range encompasses strength Classes II, III, IV and higher in sizes above 42-in. diameter.

•BECAUSE deformed cold-drawn wire has both high tensile strength for ultimate load capacity and high-bond surface qualities for crack width control, it is an efficient reinforcing material for many types of concrete structures. Several pilot test programs have demonstrated its effectiveness as reinforcing in precast concrete pipe (1) and in one-way slabs (2); however, the data developed from these programs were not sufficient to provide an accurate, dependable design method. Such a method is needed for the design of pipe, where economy of steel can be a significant factor.

The present standard for the design of most concrete pipe made in this country is "Specifications for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe" (ASTM C 76). These specifications require that pipe meet structural criteria for both crack width control and, if required, ultimate strength. These criteria, as established in the 3-edge bearing test, are (a) the load required to produce a maximum crack width of 0.01 in. (measured as defined in C 497); and (b) the load required to cause ultimate failure of the pipe. To be successful, a pipe design must reflect an accurate evaluation of performance as it relates to each of these criteria.

Previous work (3) indicates that both rigorous and semi-empirical design expressions are required to predict structural performance for all classes and sizes of pipe. For example, ultimate flexural strength, which is governed by the tensile strength of the reinforcing, can be predicted by rigorous theoretical analysis. Such formulas have been derived (3) and subsequently confirmed for a variety of pipe by considerable test data (4, 5). However, prediction of the 0.01-in. crack strength, and ultimate strength as governed by failure in diagonal tension, requires the use of semi-empirical expressions.

The semi-empirical approach is required because exact theoretical determination of these aspects of concrete behavior is beyond the present state of the art. The approach is based on physical reasoning (i. e., structural theory) which is used to determine the major variables which govern cracking and diagonal tension strength. Test data, taken over a range of the major variables, are then statistically evaluated to obtain optimized constants for the proper relationship of the governing variables. Thus, the final design equations for 0.01-in. crack load and for ultimate diagonal tension strength are semi-empirical formulas developed from a combination of structural theory and test data.

The major objective of the development effort discussed in this paper is to obtain a reliable design method for circular precast concrete pipe reinforced with welded deformed wire fabric. This includes development of basic design equations as well as an experimental test program to both implement semi-empirical theory and validate the accuracy of final design equations.

Design equations are developed which encompass the range of standard C 76 sizes and strength classes believed to have the greatest immediate potential for the use of welded deformed wire fabric. Thus, the supporting test program was designed to cover a size range from 48-in. diameter to 114-in. diameter, and a strength range from Class II to Class V (ASTM C 76). This range of sizes and strength classes also provides a good spread in the variables which are considered to be most significant in the design of pipe.

This paper is based on the results of tests on 70 pipe. Forty of these tests constitute a test program administered by the U. S. Steel Corp. during 1965-66 (6). The pipe for this program were designed by a tentative design procedure developed by Simpson Gumpertz and Heger Inc. from earlier pilot test programs on pipe with welded deformed wire fabric reinforcing (5). This design procedure was derived in part from concepts developed by Heger (3, 4) in a research program sponsored by the American Iron and Steel Institute at the Massachusetts Institute of Technology between 1958 and 1961.

The results of the remaining 30 tests were drawn from two sources. Nine of the tests were carried out by U. S. Steel in 1962 (1). The additional 21 tests were recently made by others in the concrete pipe industry (7, 8). In all cases the steel reinforcement was welded deformed wire fabric. Tables 1 and 2 indicate the variable characteristics of pipe in the foregoing test programs. Other characteristics and details of the test pipe are given later in this paper.

#### NOTATION

- $a$  = Depth of equivalent rectangular stress block at ultimate strength of concrete section for pipe with wall thickness  $5\frac{1}{2}$  in. or larger, in.
- $a'$  = Depth of equivalent rectangular stress block at ultimate strength of concrete section for pipe with wall thickness less than  $5\frac{1}{2}$  in., in.
- $A_{s1}$  = Steel area of inside cage, sq in. per ft of length
- $A'_{s1}$  = Modified steel area of inside cage, sq in. per ft of length
- $A_{s2}$  = Steel area of outside cage, sq in. per ft of length
- $A_{cs}$  = Symmetrical area of concrete surrounding each wire or rod, sq in.
- $A_v$  = Area of each line of stirrup reinforcing, sq in. per ft of length
- $b$  = Width of section
- $c$  = Correction factor, ultimate strength equations
- $C_{1,2,\dots}$  = Constants determined from test
- $C_L$  = Factor for effect of closely spaced longitudinals on diagonal tension strength, lb per ft length per ft diameter
- $d$  = Depth of section from compressive edge of concrete to center of tensile reinforcement, in.
- $d_1$  = Depth of section from compressive edge of concrete to center of inside tensile reinforcement, in.
- $d_2$  = Depth of section from compressive edge of concrete to center of outside tensile reinforcement, in.
- $D_i$  = Inside diameter of pipe, in.
- $DL_u$  = Ultimate D-Load capacity of pipe, lb per ft length per ft diameter

- $DL_{.01}$  = 0.01-in. crack D-Load capacity of pipe, lb per ft length per ft diameter  
 $D_m$  = Deformed bar or wire diameter, in.  
 $f'_c$  = Ultimate compressive strength of concrete as determined from standard 6 by 12-in. cylinders tested at age of pipe test, psi  
 $f_s$  = Tensile stress in reinforcement, psi  
 $f_{su}$  = Ultimate tensile strength of reinforcement, psi  
 $f_{su1}$  = Ultimate tensile strength of inside reinforcement, psi  
 $f_{su2}$  = Ultimate tensile strength of outside reinforcement, psi  
 $f_{s.01}$  = Stress in invert inner cage reinforcing steel at 0.01-in. crack D-Load, psi  
 $f_y$  = Yield strength (stress at 0.2 percent offset strain or 0.5 percent total strain for steel with no sharp yield point), psi  
 $h$  = Pipe wall thickness, in.  
 $L_e$  = Length of full section portion of pipe barrel between end lips  
 $L_n$  = Nominal length of pipe barrel  
 $M$  = Bending moment  
 $n$  = Exponent determined from test  
 $N_L$  = Number of wraps of inner cage reinforcing  
 $p$  = Quantity of steel as a ratio of steel to concrete area,  $A_s/bd$   
 $p_e$  = Effective reinforcement ratio,  $\pi D_m^2/4A_{cs}$   
 $R$  = Ratio of total section thickness to effective section depth (to center line of reinforcing)  
 $s$  = Circumferential spacing of each line of stirrups, in.  
 $s_L$  = Longitudinal spacing of circumferential reinforcing, in.  
 $t_b$  = Distance from tensile concrete surface to center line of reinforcing, in.  
 $V$  = Shear force, lb  
 $v$  = Nominal shear stress, psi  
 $W$  = Weight of pipe, lb per ft length  
 $w_{max}$  = Maximum crack width, in.  
 $\phi_f, \phi_{.01}$   
 $\phi_d, \phi_x$  = Variability factors

### TEST PROGRAM

The 1965-66 U.S. Steel test program was designed to evaluate the following major variables in the design equations developed from previous pilot test programs (1, 5) on precast concrete pipe reinforced with welded deformed wire fabric:

1. Welded deformed wire fabric reinforcing as compared to results obtained in previous test programs with welded smooth wire fabric (3);
2. Steel quantity;
3. Pipe diameter;
4. Pipe wall thickness; and
5. Concrete strength.

Since the results of previous pilot test programs indicate that the best possibility for immediate benefits from the use of welded deformed wire fabric is in sizes above 48-in. diameter, the test program was limited to sizes ranging between 48 and 114 in. in diameter. For this range of sizes, steel areas were varied to produce test pipes over a strength range from ASTM C 76 Class II to Class V. Variation of other design parameters was also limited to the range of values applicable to C 76 pipe.

Standard ASTM C 497 3-edge bearing tests were carried out on 40 full-scale pipe. The following firms cooperated in the manufacturing and testing: New England Concrete Pipe Corp., Price Brothers Co., and International Pipe and Ceramics Corp. In addition to the above test series which constitute the 1965-66 U.S. Steel test program, nine tests carried out in an earlier U.S. Steel pilot test program at the New England Concrete Pipe Corp. (1) are deemed as part of the overall program reviewed in this paper.

TAB  
DESCRIPTION OF TEST SPECIMEN

Pipe Mark	ASTM Class-Wall	Type Reinforcement	Nominal I, D <sub>s</sub> in.	Wall Thickness				Furnished A <sub>s1</sub> sq. in./ft.	Inner Cage Reinforcement			Steel Ult. Str. by Test psi	
				Crown in.	Invert in.	Left Springing in.	Right Springing in.		Wire Spacing in.	Wire Size # or in.	Effective d <sub>1</sub> (in.)		
											Crown	Invert	
US 48- 1a	II B	WWF Def.	48	5.00	5.00	5.00	5.00	0.140	2 × 16	D2. 4/8	3.78	3.73	98,295
US 48- 1b	II B	WWF Def.	48	4.97	4.97	5.06	5.13	0.140	2 × 16	D2. 4/8	3.88	3.76	98,295
US 48- 2a	III B	WWF Def.	48	5.19	5.16	4.97	5.06	0.185	2 × 16	D3. 2/8	4.21	4.18	96,165
US 48- 2b	III B	WWF Def.	48	5.06	5.06	5.00	5.00	0.185	2 × 16	D3. 2/8	3.84	3.96	96,165
US 48- 3a	IV B	WWF Def.	48	4.94	5.06	5.06	4.94	0.361	2 × 16	D6/6	3.92	3.93	92,627
US 48- 3b	IV B	WWF Def.	48	4.88	5.06	5.13	5.13	0.361	2 × 16	D6/6	3.68	3.93	92,627
US 48- 4a	V B	WWF Def.	48	5.13	5.22	5.06	5.16	0.720	2 × 16	D12/5	3.86	4.14	82,882
US 48- 4b	V B	WWF Def.	48	5.00	5.06	5.00	4.94	0.720	2 × 16	D12/5	3.68	3.99	82,882
US 48- 5a	IV C	WWF Def.	48	5.75	5.69	5.78	5.78	0.233	2 × 16	D4/8	4.64	4.70	90,517
US 48- 5b	IV C	WWF Def.	48	5.78	5.69	5.75	5.94	0.233	2 × 16	D4/8	4.67	4.58	90,517
US 72- 1a*	III-IV B	CD Def.	72	7.06	6.88	6.92	6.81	0.497	2-3/4	3/8	5.74	5.44	101,000
US 72- 1b*	III-IV B	CD Def.	72	7.16	7.00	6.78	6.75	0.497	2-3/4	3/8	6.15	5.69	101,000
US 72- 1c*	III-IV B	CD Def.	72	7.03	6.97	6.66	6.88	0.497	2-3/4	3/8	5.96	5.53	101,000
US 72- 2a*	III-IV B	WWF Def.	72	6.94	6.88	6.88	6.88	0.490	2 × 8	00-1/2x5	5.53	5.47	95,250
US 72- 2b*	III-IV B	WWF Def.	72	6.97	6.81	6.88	6.91	0.472	2 × 8	00-1/2x5	5.63	5.21	94,490
US 72- 2c*	III-IV B	WWF Def.	72	6.94	6.88	6.81	6.91	0.490	2 × 8	00-1/2x5	4.96	5.47	94,000
US 72- 3a*	III+ B	WWF Def.	72	6.94	6.94	6.88	6.94	0.383	2 × 8	1 × 6	5.55	5.80	78,400
US 72- 3b*	III+ B	WWF Def.	72	6.91	7.22	6.84	6.94	0.375	2 × 8	1 × 6	5.64	5.64	75,100
US 72- 3c*	III+ B	WWF Def.	72	7.00	6.94	6.91	6.91	0.378	2 × 8	1 × 6	5.68	5.42	75,550
US 72- 4a	II B	WWF Def.	72	6.97	7.03	6.81	7.00	0.236	2 × 16	D4/8	5.86	6.30	89,153
US 72- 4b	II B	WWF Def.	72	7.00	7.00	7.06	7.00	0.236	2 × 16	D4/8	6.01	6.01	89,153
US 72- 5a	III B	WWF Def.	72	6.88	7.13	6.94	7.06	0.328	2 × 16	D5/5.7	5.75	6.30	95,317
US 72- 5b	III B	WWF Def.	72	6.94	7.13	7.00	6.94	0.328	2 × 16	D5/5.7	6.25	6.19	95,317
US 72- 6a	IV B	WWF Def.	72	6.88	6.88	6.94	7.00	0.668	2 × 16	D11/5	5.69	5.56	90,837
US 72- 6b	IV B	WWF Def.	72	6.94	6.88	6.88	7.00	0.668	2 × 16	D11/5	5.69	5.56	90,837
US 72- 7a	IV B	WWF Def.	72	6.88	6.81	6.94	7.06	0.606	2 × 16	D10/5	5.64	5.51	85,943
US 72- 7b	IV B	WWF Def.	72	7.06	7.06	7.00	7.00	0.606	2 × 16	D10/5	5.82	5.75	85,943
US 72- 8a	IV B	WWF Def.	72	6.81	7.13	6.81	6.88	0.606	2 × 16	D10/5	5.57	5.97	85,943
US 72- 8b	IV B	WWF Def.	72	7.00	7.06	7.06	7.06	0.606	2 × 16	D10/5	5.69	5.71	85,943
US 72- 9a	IV B	WWF Def.	72	6.88	7.00	7.00	6.81	0.606	2 × 16	D10/5	5.60	5.69	85,943
US 72- 9b	IV B	WWF Def.	72	7.00	6.94	6.88	7.06	0.606	2 × 16	D10/5	5.82	5.53	85,943
US 72- 10a	IV B	WWF Def.	72	6.94	7.00	7.00	6.94	0.608	3 × 16	D15/5	5.41	5.65	85,363
US 72- 10b	IV B	WWF Def.	72	6.94	6.94	7.06	7.00	0.608	3 × 16	D15/5	5.47	5.66	85,363
US 72- 12a	III C+	WWF Def.	72	7.94	8.19	8.00	8.06	0.290	2 × 16	D5/7	6.88	6.62	87,250
US 72- 12b	III C+	WWF Def.	72	8.00	8.06	8.00	8.00	0.290	2 × 16	D5/7	7.06	6.81	87,250
US 72- 13a	IV C+	WWF Def.	72	8.00	7.94	7.54	8.00	0.484	2 × 16	D8/5	6.90	6.65	91,370
US 72- 13b	IV C+	WWF Def.	72	8.06	8.19	7.88	8.13	0.484	2 × 16	D8/5	6.90	6.40	91,370
US 96- 1a	II B	WWF Def.	96	9.06	9.00	8.94	8.91	0.360	2 × 16	D6/6	7.73	7.61	95,413
US 96- 1b	II B	WWF Def.	96	9.13	9.19	9.00	8.94	0.360	2 × 16	D6/6	7.99	8.05	95,413
US 96- 2a	III B	WWF Def.	96	9.00 <sup>1</sup>	9.25	8.94	9.00	0.480	2 × 16	D8/5	8.21	8.34	90,380
US 96- 2b	III B	WWF Def.	96	8.91	9.28	9.00	9.00	0.480	2 × 16	D8/5	7.88	8.12	90,380
US 96- 3a	IV B	WWF Def.	96	9.00	9.19	9.03	8.84	0.802	2 × 16	D14/5	7.92	7.92	81,876
US 96- 3b	IV B	WWF Def.	96	9.00	9.25	9.13	9.06	0.802	2 × 16	D14/5	7.67	8.16	81,876
US 96- 4a	V+ B	WWF Def.	96	8.94	9.16	8.94	9.13	1.466	2 × 16	1-D14/5 & 1-D11/SEII.	7.42	7.52	—
US 96- 5a	V+ B	WWF Def. + Stirrups	96	9.19	8.97	9.13	9.13	1.466	2 × 16	1-D14/5 & 1-D11/SEII.	7.45	6.86	83,700
US114- 1a	III B	WWF Def.	114	10.50	10.55	10.50	11.00	0.668	2 × 16	D11/5	9.18	9.24	90,837
US114- 1b	III B	WWF Def.	114	10.63	10.63	10.81	10.94	0.668	2 × 16	D11/5	9.25	9.31	90,837
US114- 2a	IV B	WWF Def.	114	10.50	10.50	10.75	10.94	1.122	2 × 16	D19/5	9.37	8.67	88,400
US114- 3a	IV+ B	WWF Def. + Stirrups	114	10.50	10.69	10.81	10.94	1.122	2 × 16	D19/5	9.00	9.44	88,400

## Notes:

- \* 1962 U. S. Steel Test Series
- 1 Assumed value
- 2 Assumed 1" cover
- 3 Combined average concrete compressive strength is average of 6x12 cylinder strengths and 0.85 x core strengths.

## Key:

- CD Def. = Cold-Drawn Wire, Deformed
- WWF Def. = Welded Wire Fabric, Deformed
- B. = Bright
- L. R. = Light Rust
- H. R. = Heavy Rust

As a further supplement to the 1965-66 U.S. Steel test program, physical data and test results on 21 additional test pipe reinforced with welded deformed wire fabric were made available by the pipe companies who performed these tests for evaluation along with the U.S. Steel test program results (7, 8). These other test pipe were manufactured and tested by the following companies: General Dynamics, Inc. (Materials Service Div.); Reliance Universal, Inc. (Concrete Products Div.) and Hannah Motors, Inc. (Kentucky Concrete Pipe Div.).

## U. S. STEEL TEST PROGRAM

Furnished As q. in./ft.	Outer Cage Reinforcement				Steel Ult. Str. by Test psi	Concrete Properties			Condition of Reinforcing	Age of Pipe at Test days	Pipe Mark
	Wire Spacing in.	Wire Size or in.	Effective $d_2$ (in.)			Cylinder Ave. Comp. Str. psi	Core Ave. Comp. Str. psi	Combined <sup>a</sup> Ave. Comp. Str. psi			
			Left Springing	Right Springing							
0.0924	4 x 16	D3.2/8	3.65	3.90	96,185	3581	5595	4051	B.	21	US 48-1a
0.0924	4 x 16	D3.2/8	4.21	4.28	96,185	2998	5370	3625	B.	21	US 48-1b
0.140	2 x 16	D2.4/8	3.83	4.10	98,295	7885	6720	7016	B.	19	US 48-2a
0.140	2 x 16	D2.4/8	3.86	3.86	98,295	7129	6915	6629	B.	21	US 48-2b
0.268	2 x 16	D4.5/8	3.94	3.82	98,430	7967	6875	7118	B.	21	US 48-3a
0.268	2 x 16	D4.5/8	3.96	4.00	98,430	7451	7315	6958	B.	21	US 48-3b
0.4848	2 x 16	D8/5	4.03	3.94	91,717	7674	7850	7273	B.	20	US 48-4a
0.4848	2 x 16	D8/5	3.96	3.72	91,717	7915	7880	7428	B.	20	US 48-4b
0.185	2 x 16	D3.2/8	4.81	4.68	96,185	7539	6270	6655	B.	19	US 48-5a
0.185	2 x 16	D3.2/8	4.65	4.84	96,185	6759	6530	6275	B.	18	US 48-5b
0.388	3-1/2	3/8	5.85	5.68	101,000	5090	6450	5250	B.	14	US 72-1a
0.388	3-1/2	3/8	5.59	5.62	101,000	5480	5790	5260	B.	14	US 72-1b
0.388	3-1/2	3/8	5.41	5.69	101,000	5900	6360	5710	B.	14	US 72-1c
0.379	2 x 8	1 x 6	6.11	5.86	75,090	5750	5600	5350	B.	14	US 72-2a
0.379	2 x 8	1 x 6	5.62	5.77	75,870	5510	6280	5440	B.	14	US 72-2b
0.379	2 x 8	1 x 6	5.73	5.83	76,500	5440	6840	5580	B.	14	US 72-2c
0.281	2 x 8	3 x 8	5.88	6.19	78,250	6360	7860	6490	B.	14	US 72-3a
0.281	2 x 8	3 x 8	5.97	6.07	75,500	5710	5950	5440	B.	14	US 72-3b
0.281	2 x 8	3 x 8	5.73	5.73	74,900	5900	5920	5560	B.	14	US 72-3c
0.191	2 x 16	D3.2/8	5.71 <sup>a</sup>	5.90 <sup>a</sup>	98,760	4757	4782	4480	L. R.	22	US 72-4a
0.191	2 x 16	D3.2/8	5.96 <sup>a</sup>	5.90 <sup>a</sup>	98,760	4713	5418	4670	L. R.	21	US 72-4b
0.236	2 x 16	D4/8	6.20	6.20	89,153	4492	4762	4314	B.	21	US 72-5a
0.236	2 x 16	D4/8	6.39	6.21	89,153	4740	5295	4644	B.	21	US 72-5b
0.242	4 x 16	D8/5	5.90	5.84	91,370	5041	4105	4420	L. R.	21	US 72-6a
0.242	4 x 16	D8/5	6.22	5.84	91,370	4935	4474	4482	L. R.	21	US 72-6b
0.416	2 x 16	D7/5	5.29	5.91	92,173	5412	5829	5229	L. R.	20	US 72-7a
0.416	2 x 16	D7/5	5.72	5.47	92,173	4784	6454	5069	L. R.	20	US 72-7b
0.416	2 x 16	D7/5	5.78	5.60	92,173	3970	5624	4294	L. R.	20	US 72-8a
0.416	2 x 16	D7/5	6.03	5.97	92,173	4704	5870	4818	L. R.	20	US 72-8b
0.416	2 x 16	D7/5	5.79	5.28	92,173	4271	4328	4034	B.	22	US 72-9a
0.416	2 x 16	D7/5	4.85	4.91	92,173	3732	3719	3504	B.	22	US 72-9b
0.403	4 x 16	D14/5	5.93	5.74	82,487	4200	6260	4648	B.	23	US 72-10a
0.403	4 x 16	D14/5	5.80	5.68	82,487	5022	6344	5170	B.	23	US 72-10b
0.191	2 x 16	D3.2/8	7.65	7.46	98,760	4643	4208	4217	B.	21	US 72-12a
0.191	2 x 16	D3.2/8	6.90	6.71	98,760	5073	5008	4747	B.	20	US 72-12b
0.354	2 x 16	D6/6	6.67	6.86	92,097	4907	6692	5219	B.	18	US 72-13a
0.354	2 x 16	D6/6	6.49	6.99	92,097	4722	5501	4704	B.	17	US 72-13b
0.236	2 x 16	D4/8	7.83	7.80	85,846	3950	5074	4096	H. R.	21	US 96-1a
0.236	2 x 16	D4/8	7.83	7.95	85,846	3820	4399	3786	H. R.	20	US 96-1b
0.322	2 x 16	D5.5/7	7.81	7.81	97,275	4787	—	4787	H. R.	19	US 96-2a
0.322	2 x 16	D5.5/7	7.81	7.62	97,275	5501	6271	5431	H. R.	23	US 96-2b
0.746	2 x 4	4/0/4	7.96	7.52	84,182	5501	7320	5681	H. R.	22	US 96-3a
0.746	2 x 4	4/0/4	7.55	7.86	84,182	5441	6916	5550	H. R.	21	US 96-3b
1.410	2 x 4	1-4/0/4 & 1-D11/5EII.	7.30	7.62	—	5630	6292	5520	H. R.	20	US 96-4a
1.320	2 x 16	1-D11/5 & 1-D11/5EII.	6.93	6.81	87,840	7140	7814	6940	L. R.	35	US 96-5a
0.480	2 x 16	D8/5	9.71	10.46	91,370	4934	4833	4604	L. R.	21	US 114-1a
0.480	2 x 16	D8/5	9.15	9.53	91,370	4935	5111	4698	L. R.	21	US 114-1b
0.805	2 x 16	D14/5	9.28	9.24	82,487	5226	—	5226	L. R.	21	US 114-2a
0.805	2 x 16	D14/5	9.73	9.86	82,487	4722	5850	4822	L. R.	21	US 114-3a

## Test Specimens

Details of the variable dimensional and physical properties measured for the actual test specimens are given in Table 1 for both the 1962 and 1965-66 U.S. Steel test series. The following standard details were specified for these test specimens:

1. Arrangement of specimen: Circular ring with flat ends (no tongue-and-groove), 4 ft in length.

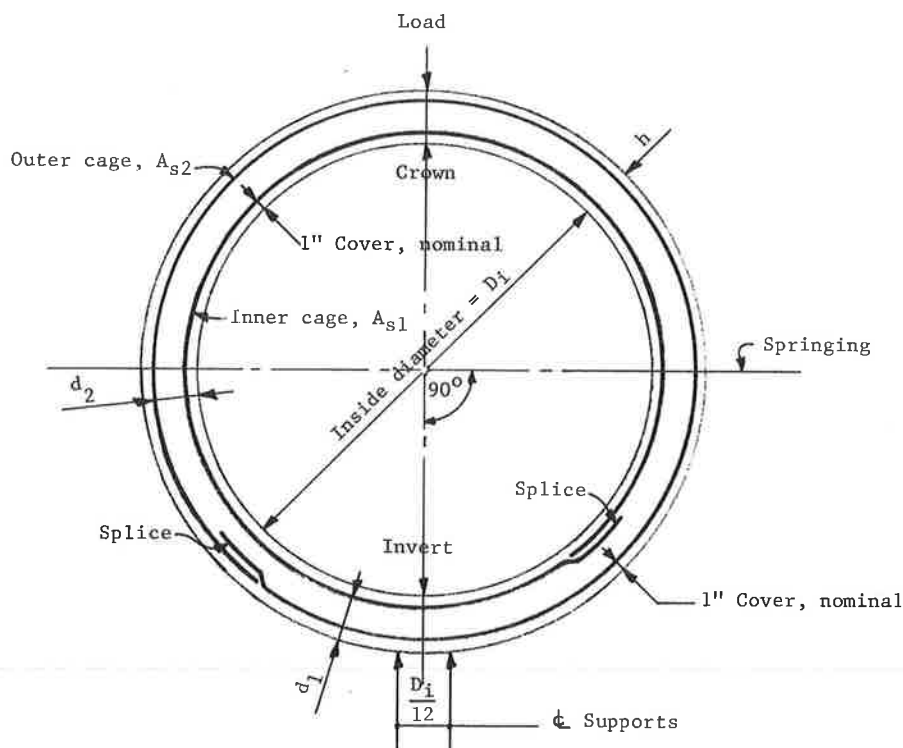
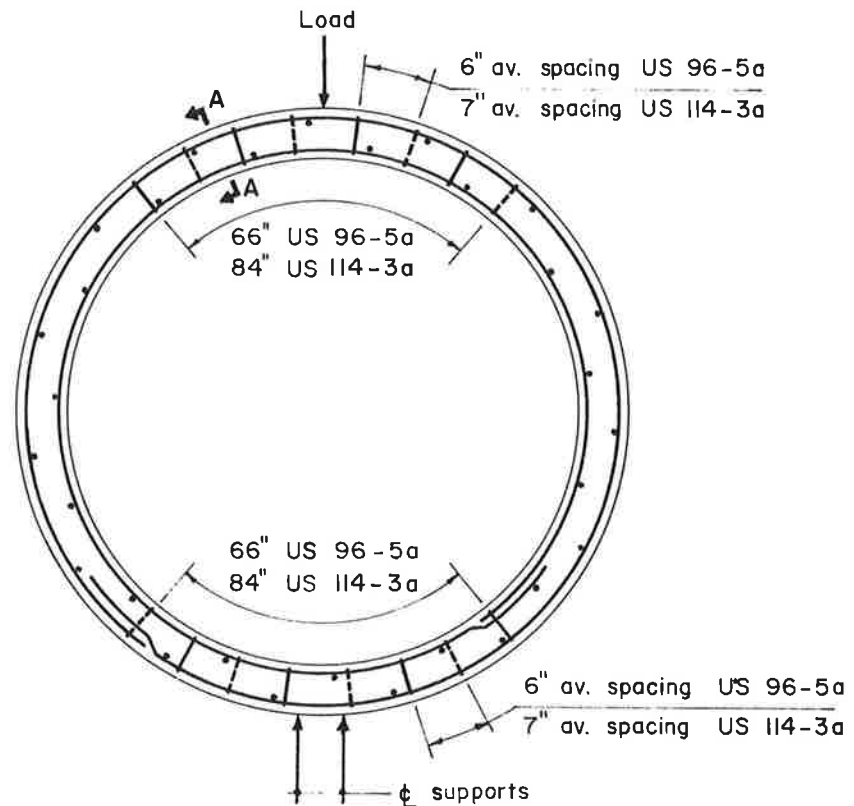


Figure 1. Pipe test specimens—typical transverse section.

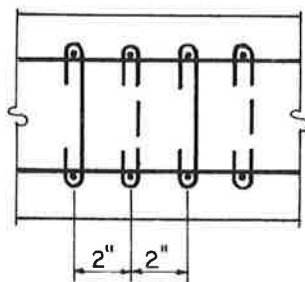
2. Reinforcement layout and nominal cover: As shown in Figure 1.
3. Type of reinforcing: Welded deformed steel wire and wire fabric (ASTM A 496-64 and A 497-64). Wire reinforcing size and spacing as in Table 1.
4. Method of pipe manufacture: Cast process. Cast vertically in steel forms with concrete placed at a rate of rise of approximately 6 in. per min and vibrated either with internal or external vibrators. Begin steam curing no sooner than 4 hr after casting and limit maximum rate of temperature increase to 15 F per hr. Steam cure at average temperature of 130 F for approximately 12 hr (5 to 6 hr in forms, 6 to 7 hr after forms are stripped). Provide enclosure over both inside and outside of pipe to permit steam to contact both inside and outside surfaces of the pipe.
5. Stirrup arrangement for pilot tests US 96-5a and US 114-3a: As shown in Figure 2.

Two nearly identical specimens were fabricated for each variation of a parameter. Specimens were designed to meet ASTM C 76 strength requirements for Class II, III, IV, or V pipe, respectively, as indicated in Table 1. Steel areas were determined using the design procedure developed from previous pilot tests on pipe with welded deformed wire fabric (5). This procedure assumes nominal or "design" steel areas, pipe dimensions, steel location and concrete strength. No allowance was made for manufacturing variability in the design. Wall thicknesses were either standard ASTM C 76 Wall B or C. Note, however, that for some nominal Wall C specimens, thickness values differ slightly from standard C 76 values because of limitations of available forms. Two specimens were fabricated with stirrups to serve as pilot tests for direction in possible future tests of higher D-Load pipe (Fig. 2).

Test specimens were inspected during manufacture at a time just before and during placement of concrete. Steel size and location and form dimensions were checked prior to placement of concrete in the forms. Also, the existence of rust on the reinforcing



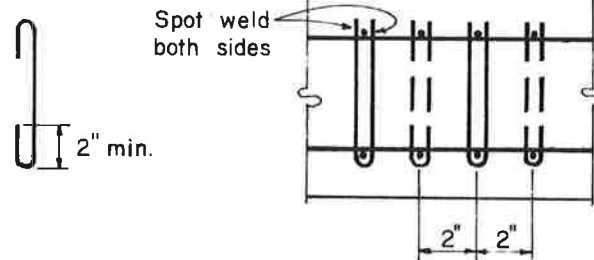
D10 stirrups at 12" o.c. each line.  
Stagger stirrups on adjacent lines  
to produce 6" average spacing.



**SECTION A-A US 96-5a**

AREA = 0.30 in<sup>2</sup>/ft. at 6" average  
Circular spacing.

Double D5 stirrups at 14" o.c.  
each line. Stagger stirrups on  
adjacent lines to produce 7"  
average spacing.



**SECTION A-A US 114-3a**

AREA = 0.30 in<sup>2</sup>/ft. at 7" average  
Circular spacing.

Figure 2. Stirrup details.

## DESCRIPTION OF TEST SPECIMENS

Pipe Mark	ASTM Class-Wall	Type Reinforcement	Nominal I. D. in.	Wall Thickness				Furnished $A_{st}$ sq. in./ft.	Wire Spacing in.	Inner Cage Reinforcement	
				Crown in.	Invert in.	Left Springing in.	Right Springing in.			Wire Size # or in.	Effective Crown
RU-KP 108-1a	III-IV B	WWF Def.	108	10.00	10.00	10.00	10.00	0.777	2 x 12	D13/	9.00
RU-KP 108-1b	III-IV B	WWF Def.	108	10.13	10.13	10.13	10.13	0.777	2 x 12	D13/	8.75
RU-KP 108-2	III-IV B	WWF Def.	108	10.00	10.00	10.00	10.00	0.777	2 x 12	D13/	8.88
RU-KP 108-3	III B	WWF Def.	108	10.13	10.13	10.13	10.13	0.567	2 x 12	D9.5/	8.88
MS 72-1a	III B	WWF Def.	72	7.00	7.00	7.00	7.00	0.354	2 x 16	D6/6	5.86
MS 72-1b	III B	WWF Def.	72	7.00	7.00	7.00	7.00	0.354	2 x 16	D6/6	5.36
MS 72-1c	III B	WWF Def.	72	7.00	7.00	7.00	7.00	0.354	2 x 16	D6/6	5.61
MS 84-1a	III B	WWF Def.	84	8.00	8.00	8.00	8.00	0.416	2 x 16	D7/5	6.72
MS 84-1b	III B	WWF Def.	84	8.00	8.00	8.00	8.00	0.416	2 x 16	D7/5	7.04
MS 84-1c	III B	WWF Def.	84	8.00	8.00	8.00	8.00	0.416	2 x 16	D7/5	6.85
MS 96-1a	III B	WWF Def.	96	9.00	9.00	9.00	9.00	0.484	2 x 16	D8/5	7.71
MS 96-1b	III B	WWF Def.	96	9.00	9.00	9.00	9.00	0.484	2 x 16	D8/5	8.09
MS 96-1c	III B	WWF Def.	96	9.00	9.00	9.00	9.00	0.484	2 x 16	D8/5	7.96
MS 96-2a	III-IV B	WWF Def.	96	9.13	9.13	9.13	9.13	0.570	2 x 12	D9.5/5	7.90
MS 96-2b	III-IV B	WWF Def.	96	9.00	9.00	9.00	9.00	0.570	2 x 12	D9.5/5	7.70
MS 96-2c	III-IV B	WWF Def.	96	9.00	9.00	9.00	9.00	0.570	2 x 12	D9.5/5	7.84
MS 114-1a	III A	WWF Def.	114	9.50	9.50	9.50	9.50	0.668	2 x 16	D11/5	7.68
MS 114-1b	III A	WWF Def.	114	9.50	9.50	9.50	9.50	0.668	2 x 16	D11/5	9.12
MS 114-1c	III A	WWF Def.	114	9.50	9.50	9.50	9.50	0.668	2 x 16	D11/5	8.00
MS 114-2a	III+ A	WWF Def.	114	9.63	9.50	9.50*	9.50*	0.780	2 x 12	D13, 1/4	8.43
MS 114-2b	III+ A	WWF Def.	114	9.50	9.50	9.50*	9.50*	0.780	2 x 12	D13, 1/4	8.80

\* Assumed Values

steel was noted, if present. A record was made of the method of curing, curing temperatures and other details pertinent to the manufacturing process.

Details of the variable dimensional and physical properties measured for the actual test specimens in the Materials Service test series and the Reliance Universal-Kentucky Concrete Pipe test series are given in Table 2. Standard details for both of these test series were similar to those for the U. S. Steel test series, except that the specimens were 7 ft 6 in. or 8 ft 0 in. in length with standard tongue and lip ends.

#### Material Control Tests

Control tests were carried out to determine significant structural properties of steel and concrete materials in the U. S. Steel test pipes. Ultimate tensile strength, and stress at 0.2 percent offset strain and at 0.5 percent total strain, were obtained from samples cut from each sheet of welded wire fabric used in the test program. Concrete compressive strengths were obtained from results of tests on both standard cylinders and cores cut from the wall of the pipe after test. Three standard 6 by 12-in. cylinders were made and stored with each test pipe. Two 4-in. diameter cores were cut from the wall (in the region of the quarter points of the ring) of each test pipe within approximately 24 hr of the time of test. Compressive strength was estimated by averaging the cylinder test results and 0.85 times "corrected" (i. e., corrected for length-diameter ratio in accordance with ASTM C 42) core strengths. Previous experience indicates that corrected core strengths do not agree with cylinder strengths and that cylinder strengths average about 0.85 times corrected core strengths.

Concrete strengths for the Materials Service test specimens were obtained from an average of 6 standard cylinder tests for each specimen. Standard cylinder test results were the basis for concrete strength values for the Reliance Universal-Kentucky Concrete Pipe tests.

#### Test Procedure

For the U. S. Steel test program, test pipe were all loaded in standard 3-edge bearing machines located at the plants of the three companies participating in the test program.



## IS—OTHER TEST PROGRAMS

Steel Ult. Str. by Test psi	Furnished A <sub>52</sub> sq. in./ft.	Wire Spacing in.	Outer Cage Reinforcement		Steel Ult. Str. by Test psi	Age of Pipe at Test days	Cylinder Ave. Comp. Str. psi	Pipe Mark	
			Wire Size # or in.	Effective d <sub>2</sub> (in.)					
				Left Springing					Right Springing
94,700	0.594	2 x 12	D10/	8.75	8.75	97,000	4850	RU-KP 108-1a	
94,700	0.594	2 x 12	D10/	9.00	9.00	97,000	5350	RU-KP 108-1b	
94,700	0.594	2 x 12	D10/	8.75	8.75	97,000	5800	RU-KP 108-2	
90,000	0.430	2 x 12	D10/	8.88	8.88	90,000	5350	RU-KP 108-3	
92,097	0.267	2 x 16	D4.5/8	5.38	5.38	98,430	7180	MS 72-1a	
92,097	0.267	2 x 16	D4.5/8	5.38	5.38	98,430	7480	MS 72-1b	
92,097	0.267	2 x 16	D4.5/8	5.13	5.13	98,430	7070	MS 72-1c	
92,173	0.290	2 x 16	D5/7	6.37	6.37	87,250	7730	MS 84-1a	
92,173	0.290	2 x 16	D5/7	6.87	6.87	87,250	7780	MS 84-1b	
92,173	0.290	2 x 16	D5/7	7.12	7.12	87,250	7305	MS 84-1c	
91,370	0.354	2 x 16	D6/6	7.36	7.36	92,097	6985	MS 96-1a	
91,370	0.354	2 x 16	D6/6	7.86	7.86	92,097	8135	MS 96-1b	
91,370	0.354	2 x 16	D6/6	8.11	8.11	92,097	7250	MS 96-1c	
83,000	0.432	2 x 12	D7.2/5	7.98	7.98	93,500	5185	MS 96-2a	
83,000	0.432	2 x 12	D7.2/5	7.85	7.85	93,500	4213	MS 96-2b	
83,000	0.432	2 x 12	D7.2/5	7.85	7.85	93,500	4127	MS 96-2c	
90,837	0.484	2 x 16	D8/5	7.59	7.59	91,370	7410	MS 114-1a	
90,837	0.484	2 x 16	D8/5	7.34	7.34	91,370	7180	MS 114-1b	
90,837	0.484	2 x 16	D8/5	7.34	7.34	91,370	7835	MS 114-1c	
97,000	0.590	2 x 12	D9.9/5	8.32	8.32	86,000	4880	MS 114-2a	
97,000	0.590	2 x 12	D9.9/5	8.32	8.32	86,000	4693	MS 114-2b	

Test procedure followed the requirements and test set-up given in ASTM C 497-64T. Testing was carried out under the direct supervision of an engineer of the participating companies, and an engineer representing U.S. Steel. All 0.01-in. crack measurements were made by both the pipe company engineer and the U.S. Steel engineer.



Figure 3. Test set-up for standard 3-edge bearing pipe.

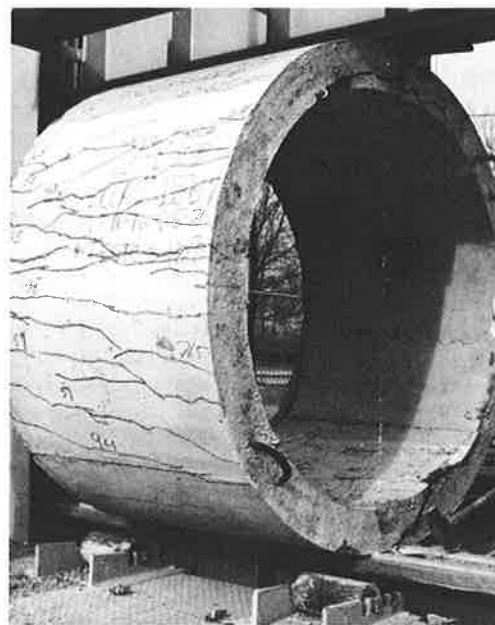


Figure 4. Crack pattern and mode of failure for 72-in. pipe.



Figure 5. Ultimate failure by diagonal tension in 72-in. pipe.

Test specimens were loaded to failure at an approximate rate of 8000 lb per min (2000 lb per ft length per min). Load was recorded at the occurrence of the first visible crack and the 0.01-in. crack. The 0.01-in. crack was determined in accordance with ASTM C 497, using a standard leaf gage. The load at the formation of each crack occurring at the crown and invert was also noted, and the number, spacing, and pattern of these cracks were recorded. Finally, the failure load and the mode of failure were noted. Photographs were taken of the test set-up, the crack patterns, and the mode of failure (Figs. 3, 4, and 5).

Deformation of the pipe was also determined during the test. Vertical and horizontal diameters of the pipe were measured by tape at zero load, at the 0.01-in. crack load, and at 1.4 times the specified C 76 0.01-in. crack load.

At the completion of each test, the concrete covering the inner cage at crown and invert and the outer cage at springings was broken off in several small areas, and the depth of cover was measured. Overall wall thickness was measured at each end of the pipe at the crown, invert and springings.

Essentially the same procedure was used for tests in the Materials Service and the Reliance Universal-Kentucky Concrete Pipe programs, except that data on deflection and crack spacing were not recorded for all of these tests.

#### Discussion of Results

Principal results from the U. S. Steel test program are summarized in Table 3. Results from the Materials Service and Reliance Universal-Kentucky Pipe test programs

TABLE 3  
TEST RESULTS—STEEL TEST PROGRAM

Pipe Mark	1st Visible Crack D-Load	D-Load 0.01" Crack Design <sup>a</sup>	Crack Test	$\frac{DL_{0.01 \text{ test}}}{DL_{0.01 \text{ design}}}$	D-Load Ultimate Design <sup>a</sup>	Test	$\frac{DL_{u \text{ test}}}{DL_{u \text{ design}}}$	Average Crack Spacing at 0.01" Crack Load (in.)		Mode of Failure
								Crown	Invert	
US 48-1a	1133	1000	1281	1.28	1500	1975	1.32	4	—	F.
US 48-1b	1096	1000	1330	1.33	1500	2025	1.35	5	4	F.
US 48-2a	997	1350	1664	1.23	2000	2629	1.32	4	4-1/2	F.
US 48-2b	1318	1350	1664	1.23	2000	2672	1.34	3	4-1/2	F.
US 48-3a	1490	2000	2385	1.19	3000	3112	1.04	3-1/2	3	D. T.
US 48-3b	1244	2000	2385	1.19	3000	3125	1.04	3	5	D. T.
US 48-4a	1553	3000	3074	1.03	3750	3978	1.06	5	4-1/2	D. T.
US 48-4b	1676	3000	3651	1.22	3750	4095	1.09	2-1/2	4	D. T.
US 48-5a	1398	2000	2242	1.12	3000	3835	1.28	4-1/2	4-1/2	F., R. T.
US 48-5b	1614	2000	2373	1.19	3000	3750	1.25	5	4-1/2	F., R. T.
US 72-1a	825	1800	1885	1.05	2700	3080	1.14	4	4	D. T., R. T.
US 72-1b	825	1800	2050	1.14	2700	2940	1.09	6	6	D. T.
US 72-1c	780	1800	1520	0.84	2700	2970	1.10	8	4-1/2	D. T.
US 72-2a	827	1800	1825	1.01	2700	2990	1.11	5	5-1/4	D. T.
US 72-2b	555	1800	1735	0.96	2700	2550	0.94	5	7	D. T.
US 72-2c	692	1800	1650	0.92	2700	2740	1.01	5	8	D. T.
US 72-3a	508	1500	1955	1.30	2250	2520	1.12	8-1/2	7-1/2	F.
US 72-3b	646	1500	1520	1.01	2250	2560	1.14	7-1/2	5-1/2	F.
US 72-3c	780	1500	1650	1.10	2250	2615	1.17	7-1/2	5-1/2	F., D. T.
US 72-4a	958	1000	1542	1.54	1500	2000	1.33	5-1/2	4-1/2	F., R. T.
US 72-4b	875	1000	1313	1.31	1500	1958	1.31	4-1/2	8	F.
US 72-5a	875	1350	1417	1.05	2000	2000	1.00	6-1/2	8	D. T.
US 72-5b	958	1350	1458	1.08	2000	2188	1.09	7	6-1/2	R. T.
US 72-6a	1083	2000	1958	0.98	3000	2875	0.96	6	6	D. T.
US 72-6b	1083	2000	2250	1.13	3000	2729	0.91	5-1/2	3-1/2	R. T.
US 72-7a	1125	2000	2458	1.23	3000	2813	0.94	5-1/2	8-1/2	D. T.
US 72-7b	1000	2000	2500	1.25	3000	2791	0.93	6	6	D. T.
US 72-8a	1042	2000	1958	0.98	3000	2375	0.79	5	6-1/2	D. T.
US 72-8b	1042	2000	2333	1.17	3000	2604	0.87	6	6	D. T.
US 72-9a <sup>a</sup>	875	2000	1583	0.79 <sup>a</sup>	3000	2000	0.67 <sup>a</sup>	6	4	D. T.
US 72-9b <sup>a</sup>	917	2000	1208	0.60 <sup>a</sup>	3000	2313	0.77 <sup>a</sup>	5	6	R. T., D. T.
US 72-10a	917	2000	2458	1.15	3000	2750	0.92	7-1/2	6	D. T.
US 72-10b	917	2000	2458	1.23	3000	4042	1.35	3-1/2	6	D. T.
US 72-12a	1000	1350	1458	1.08	2000	2250	1.13	7-1/2	6	D. T.
US 72-12b	1167	1350	1667	1.24	2000	2250	1.13	6	6-1/2	D. T.
US 72-13a	1333	2000	2500	1.25	3000	3208	1.07	6	6	D. T.
US 72-13b	1000	2000	1938	0.97	3000	2458	0.82	6	6-1/2	D. T.
US 96-1a	653	1000	1560	1.56	1500	1780	1.19	4-1/2	3-3/4	F., R. T.
US 96-1b	622	1000	1490	1.49	1500	1785	1.20	5	6-1/2	F., R. T.
US 96-2a	1028	1350	1685	1.25	2000	2180	1.09	—	5-1/2	F., R. T.
US 96-2b	1059	1350	1685	1.25	2000	2355	1.18	—	5	R. T., D. T.
US 96-3a	1247	2000	2500	1.25	3000	3200	1.06	4	4	D. T., R. T.
US 96-3b	1403	2000	2650	1.32	3000	3090	1.03	4	5	R. T.
US 96-4a	1653	3000	3530	1.18	3750	4470	1.19	4-1/2	4	D. T.
US 96-5a <sup>a</sup>	1684	3000	3052	1.02	3750	4820 <sup>1,2</sup>	1.29 <sup>1,2</sup>	4	4-1/2	
US114-1a	789	1350	1579	1.17	2000	1908	0.95	5-1/2	8	R. T., D. T.
US114-1b	895	1350	1553	1.15	2000	2184	1.09	8-1/2	7	R. T.
US114-2a	921	2200	2105	0.96	3300	2329	0.71	4-1/2	6	R. T.
US114-3a <sup>a</sup>	1158	2200	2632	1.19	3300	4552 <sup>a</sup>	1.37 <sup>a</sup>	5-1/4	5-3/4	R. T.

**Notes:**

- Capacity of testing machine reached at  $(DL)_u = 4816$ .
- This was a special test for low concrete strength.
- Stirrup reinforcing used.
- "Design 0.01 inch crack D-Load" is D-Load obtained using tentative design procedure for 0.01 inch crack, reference 2, with nominal pipe dimensions, concrete strength and steel areas without allowance for manufacturing tolerance.
- Design ultimate D-Load is 1.5 times design 0.01 inch crack D-Load.

**Key:**

- F. = Flexural Failure  
D. T. = Diagonal Tension Failure  
R. T. = Radial Tension Failure

TABLE 4  
TEST RESULTS—OTHER TEST PROGRAMS

Pipe Mark	D-Load .01" Crack Design <sup>1</sup>	Test	DL .01 test DL .01 design	D-Load Ultimate Design <sup>2</sup>	Test	DL <sub>u</sub> test DL <sub>u</sub> design	Mode of Failure
I. Materials Service Corporation Tests							
MS 72-1a	1350	2040	1.51	2000	2830	1.41	D. T.
MS 72-1b	1350	2250	1.67	2000	3000	1.50	D. T.
MS 72-1c	1350	2250	1.67	2000	2565	1.28	D. T.
MS 84-1a	1350	1750	1.30	2000	2190	1.09	D. T.
MS 84-1b	1350	1570	1.16	2000	2190	1.09	D. T.
MS 84-1c	1350	2380	1.76	2000	2580	1.29	D. T.
MS 96-1a	1350	1850	1.37	2000	2330	1.16	D. T.
MS 96-1b	1350	1530	1.13	2000	2050	1.03	D. T.
MS 96-1c	1350	1380	1.02	2000	2130	1.06	D. T.
MS 96-2a	1500	1783	1.19	2250	2233	0.99	D. T., R. T.
MS 96-2b	1500	1950	1.30	2250	2135	0.95	D. T.
MS 96-2c	1500	1920	1.28	2250	2170	0.97	D. T., R. T.
MS 114-1a	1350	1620	1.20	2000	2460	1.23	D. T.
MS 114-1b <sup>3</sup>	1350	1620	1.20	2000	1900	0.95 <sup>a</sup>	D. T.
MS 114-1c	1350	1650	1.22	2000	2260	1.13	D. T.
MS 114-2a	1400	1610	1.15	2100	1660	0.79	D. T., R. T.
MS 114-2b	1400	1430	1.02	2100	1980	0.94	D. T., R. T.
II. Reliance Universal, Kentucky Concrete Pipe							
RU-KP 108-1a	1650	2220+	1.34+	2450	2220	0.91	R. T.
RU-KP 108-1b	1650	2200+	1.33+	2450	2200	0.90	D. T.
RU-KP 108-2 <sup>4</sup>	1650	2080	1.26	2450	2930	1.20 <sup>4</sup>	R. T., D. T.
RU-KP 108-3	1350	1390	1.03	2000	2105	1.06	R. T., D. T.

Notes:

1. "Design 0.01 inch crack D-Load" is D-Load obtained using tentative design procedure for 0.01 inch crack, reference 2, with nominal pipe dimensions, concrete strength and steel areas with an allowance for manufacturing tolerance of about 10%.
2. Design ultimate D-Load is 1.5 times design 0.01 inch crack D-Load.
3. Pipe had only 1/4" cover at invert.
4. Stirrup reinforcing used.

Key:

- D. T. = Diagonal Tension Failure  
R. T. = Radial Tension Failure

are given in Table 4. Strength results are shown in terms of D-Load strength, which is defined as the total load per foot of pipe length in the 3-edge bearing test divided by the nominal pipe inside diameter in feet.

A comparison of test D-Loads and "design" D-Loads is shown in Figures 6 and 7. The design D-Loads are the D-Load strengths determined by the tentative design procedure developed in an earlier pilot test program (5). As noted previously, this procedure was used to design the test specimens. Note that no allowance for manufacturing variability was considered in determining steel areas for the 1965-66 U.S. Steel test specimens. The pipe specimens in the 1962 U.S. Steel program had steel areas arbitrarily selected, based on current C 76 requirements or a fixed percent reduction from current requirements.

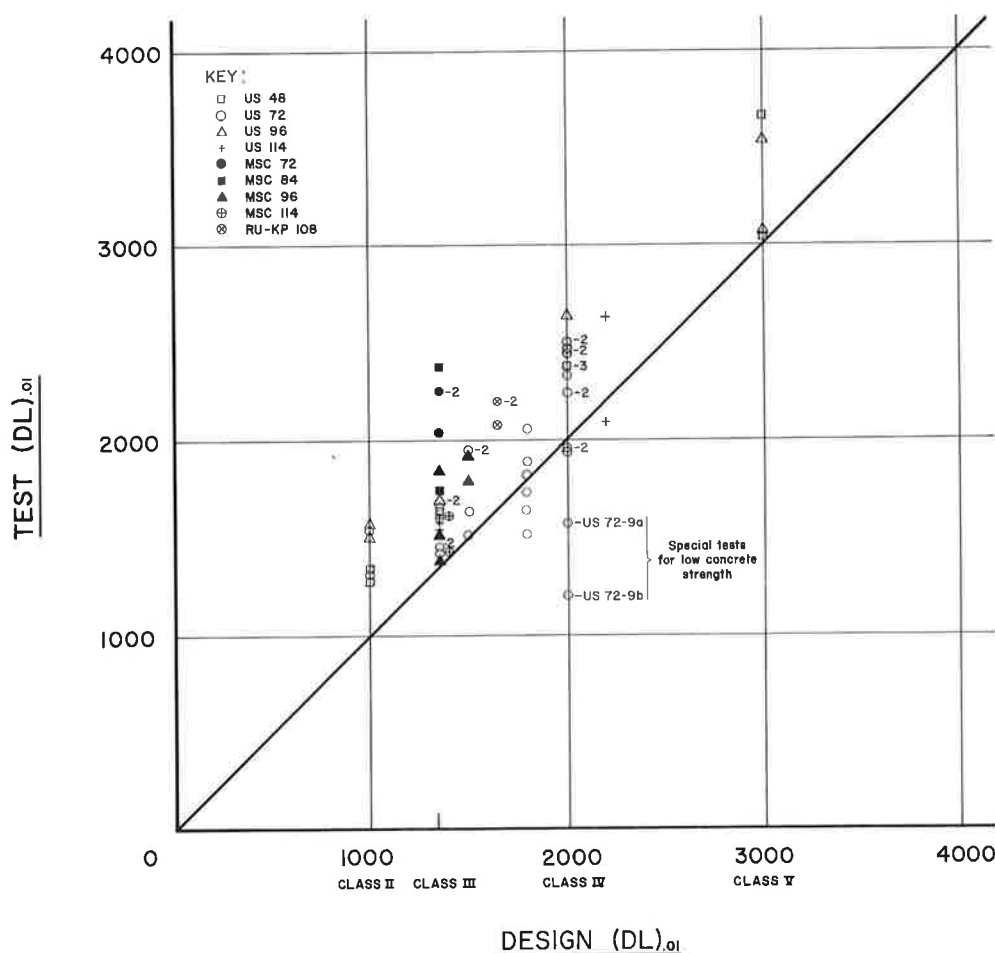


Figure 6. Comparison of test and required C 76 0.01-in. crack strengths.

The pipe in the other test programs were production pipe. For this reason design D-Load was determined by the above tentative design procedure, assuming approximately a 10 percent increase in steel areas for manufacturing variability. Most of these test pipe were designed as Class III pipe.

The test results, when compared with design strengths given by the previous tentative design equations (5) based on nominal properties of the pipe, indicate that:

1. The previous tentative design equations adequately predict 0.01-in. crack strength and ultimate flexural strength. Even though steel areas were proportioned with no allowance for manufacturing variability, nearly all specimens reached the design 0.01-in. crack strength. All specimens which failed by flexure reached their required ultimate strength.

2. For certain classes of pipe, the tentative design equations do not adequately predict ultimate strength in diagonal tension. The test strengths were lower than the tentative design equations would predict for a few of the large Class III test pipe and a number of the Class IV specimens.

There are three probable factors which contribute to the disagreement between the design diagonal tension strengths as determined by the pilot test methods (5) and the ultimate values obtained during the tests. These are:

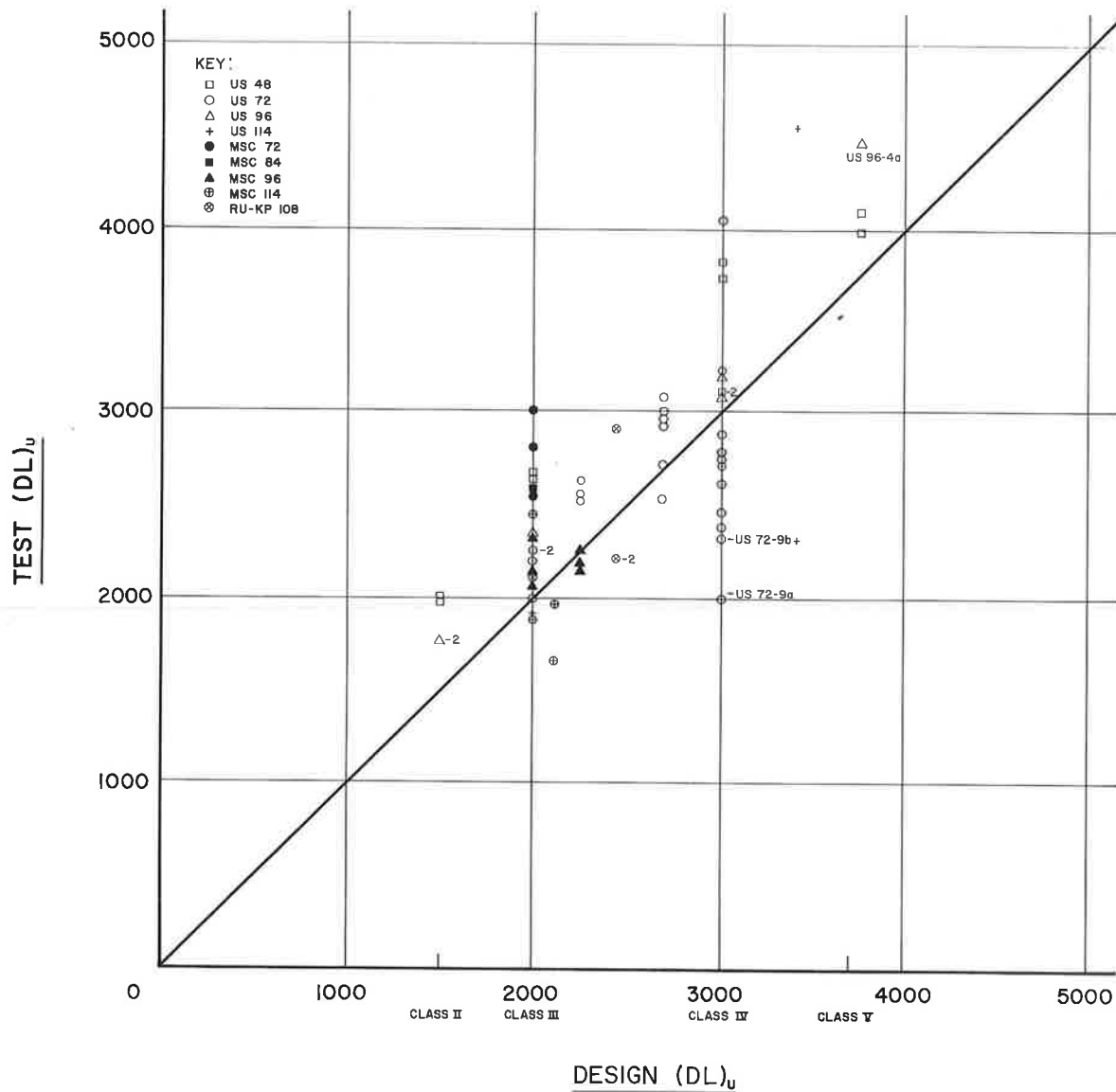


Figure 7. Comparison of test and required C 76 ultimate strengths.

1. Size effect: Test results show a relatively lower diagonal tension strength as pipe diameter increases. This size effect was not apparent from earlier pilot tests (5) and MIT tests (4); therefore, it was not included in the previous diagonal tension equation. Most of the pipe in these programs were 72 in. in diameter. Only a few specimens were included at the extremes of the range from 48 to 108 in. in diameter.

2. Influence of longitudinals: Wide spacing of longitudinals on welded deformed wire fabric reinforcing may contribute to a reduction in diagonal tension strength from that of pipe reinforced with welded wire fabric having longitudinals spaced less than 16 in. The test program was not designed to investigate the parameter of spacing of longitudinals; to define the quantitative extent of this influence, further tests are necessary.

3. Effect of close flexural crack spacing: Close flexural crack spacing, which results from the better bond achieved with deformed wire, may cause a slight reduction

in diagonal tension strength over pipe reinforced with welded smooth wire fabric. This factor is difficult to isolate and define quantitatively because of the inherent variability associated with both flexural crack spacing and diagonal tension failure of concrete.

The effect of these factors on the development of a design theory for concrete pipe is discussed next.

### THEORY OF STRUCTURAL BEHAVIOR

A dependable design method for concrete pipe, to meet structural performance criteria given in ASTM C 76, must be based on an accurate understanding of pipe structural behavior. Since performance criteria in ASTM C 76 are based on 3-edge bearing test strengths, 3-edge bearing test results are a direct indication of the adequacy of structural performance of a given design. However, much less testing will be required, and the strength test data that are obtained will be much more valuable, if limited test results can be translated into a general scheme for rational evaluation of pipe structural behavior. Such a theory for pipe structural behavior is discussed here. As indicated previously, for ultimate flexural strength, a rigorous theoretical analysis yields practical results, while for 0.01-in. crack strength and for ultimate diagonal tension strength, a combination of physical reasoning to determine major variables and statistical evaluation of test data to obtain the quantitative relationship of variables is required.

A previous investigation of the structural behavior of concrete pipe carried out in 1958-61 at MIT (3) resulted in methods for calculating 0.01-in. crack strength, ultimate flexural strength, and ultimate diagonal tension strength of concrete pipe with welded smooth wire fabric reinforcing. The concepts and findings of this work provide a partial basis for the development of the design theory for pipe with welded deformed wire fabric.

#### Control of Crack Width

The pipe industry has long recognized the importance of crack control in concrete pipelines. This is reflected in ASTM C 76 which established a specified test load to produce a maximum crack width no greater than 0.01-in. as an important criterion of structural performance.

For some classes of pipe and types of reinforcing, the required amount of steel is governed by this criterion for control of cracking. Thus, to be effective, high strength reinforcing, such as cold-drawn wire, should also possess commensurate good crack control qualities. Deformations have been introduced on cold-drawn wire in an attempt to achieve this high degree of crack control. These deformations improve bond between steel and concrete, thus producing a close spacing of fine cracks, rather than a smaller number of wider cracks.

#### Steel Stress at 0.01-In. Crack Load

The 0.01-in. crack strength is closely related to the stress in the inner reinforcing steel at crown and invert. A fairly accurate estimate of this stress can be obtained from the well known "thin ring" elastic analysis. The invert stress is slightly higher than the stress at the crown because the effect of pipe weight is greater at the invert.

The following formula for stress in inner cage reinforcing at invert is obtained from an elastic "thin ring" analysis for 3-edge bearing type load (5):

$$f_{s.01} = \frac{0.014 D_i^2 \left( DL_{.01} + 9 \frac{W}{D_i} \right)}{A_{s1} d_1} \quad (1)$$

Equation 1 provides the means to translate 0.01-in. crack 3-edge bearing load into steel stress values. Various relationships between steel stress and crack width which have been developed by others are discussed below.

### Relationships for Crack Width

Because concrete is not a homogeneous material, and because of additional complexities introduced by the composite nature of reinforced concrete, crack formation and crack width are necessarily subject to considerable variability in precast concrete pipe, even under closely controlled conditions of manufacture. Because of this problem of variability, it is not possible to make a quantitative evaluation of all factors which affect cracking in reinforced concrete. Nevertheless, in recent years considerable progress has been made toward the development of semi-empirical expressions for quantitative estimates of crack width with certain types of reinforcing steels. Three such expressions are presented here for later use in evaluation of data obtained in this investigation.

PCA (9), based on A305 type deformed bars:

$$w_{\max} = 11.5 \times 10^{-8} f_s \sqrt[3]{A_{cs}} \quad (2)$$

Cornell University (10), based on A305 type deformed bars:

$$w_{\max} = 9.1 \times 10^{-8} R (f_s - 5000) \sqrt[3]{t_b A_{cs}} \quad (3)$$

CEB (9), based on European type deformed bars:

$$w_{\max} = 2.1 \times 10^{-8} f_s \left( \frac{D_m}{p_e} \right) \left( \frac{1}{4.5 p_e + 0.40} \right) \quad (4)$$

These expressions provide useful insight into the major variables which influence cracking of reinforced concrete flexural members. They all reflect the results of extensive tests and careful analysis of test data by a number of researchers who are working on this general problem of crack width determination for reinforced concrete design.

### Development of 0.01-In. Crack Strength Equation

Crack control qualities of deformed cold-drawn wire have been studied by means of pull-out and slab tests (2). The results of this work indicate that deformed wire may have bonding qualities similar to A305 deformed bars. Thus, the PCA cracking formula was investigated on a first trial basis for evaluating 0.01-in. crack strength of pipe with deformed cold-drawn wire. The following formula for 0.01-in. crack D-Load strength follows directly from Eqs. 1 and 2:

$$DL_{.01} = \frac{6.2 \times 10^6 A_{s1} d_1}{\sqrt[3]{A_{cs}} D_i^2} - 9 \frac{W}{D_i} \quad (5)$$

This formula does not recognize the ability of the concrete between cracks to carry tension forces which can be significant in lightly reinforced pipe. Inclusion of a term to account for tensile resistance provided by concrete between cracks results in the following equation:

$$DL_{.01} = C_1 \sqrt{f_c} + \frac{C_2 A_{s1} d_1}{\sqrt[3]{A_{cs}} D_i^2} - 9 \frac{W}{D_i} \quad (6)$$

This theoretical equation can be used to calculate the 0.01-in. crack D-Load strength provided the constants are determined from proper test data. Test data from the several test programs with pipe reinforced with welded deformed wire fabric are plotted in terms of the above parameters in Figure 8. The constants  $C_1$  and  $C_2$  are determined



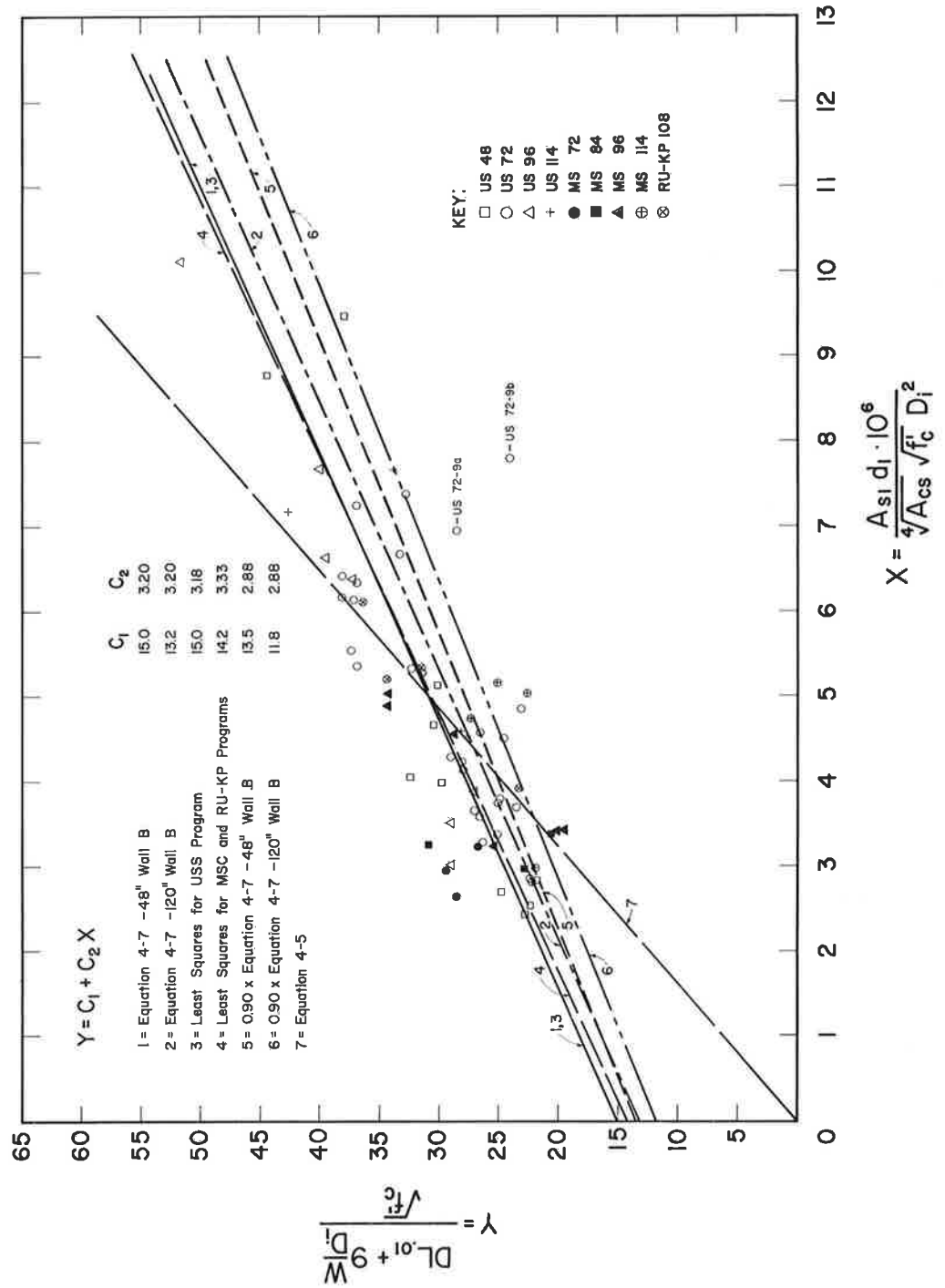


Figure 8. Development of 0.01-in. crack D-Load strength equation.

by a least squares analysis of the data in the U.S. Steel program only. They are:

$$\begin{aligned} C_1 &= 180 \\ C_2 &= 3.18 \times 10^6 \end{aligned}$$

$C_1$  can be further modified to recognize a small variation in the relative area of concrete between cracks which occurs because of the three different ratios of wall thickness to pipe diameter used for C 76 pipe. If this is done, the following semi-empirical equation for evaluation of 0.01-in. crack strength behavior is obtained:

$$DL_{.01} = \frac{144 h \sqrt{f'_c}}{D_i} + \frac{3.2 \times 10^6 A_{s1} d_1}{\sqrt{A_{cs}} D_i^2} - 9 \frac{W}{D_i} \quad (7)$$

Equation 7 is very similar to the  $DL_{.01}$  equation proposed earlier (1, 5) with a change in the units of  $D_i$  from feet to inches and smaller changes in the constants  $C_1$  and  $C_2$ . It is predicated on a relationship between the crack width and the elongation of the reinforcing steel over the length between cracks. Consequently, this equation can be used only for steels which produce crack spacing similar to those in the test program (i.e., deformed reinforcing). Furthermore, it should be used only where crack patterns extend over sufficient lengths to have a normal crack spacing on each side of the 0.01-in. crack. This means three or more cracks should be formed at the crown or invert at the 0.01-in. crack load level. For pipes less than 48 in. in diameter, only one or two cracks may form at the crown or invert; hence, Eq. 7 may not give accurate results for pipe in this size range. However, as a practical matter, the 0.01-in. crack criterion probably does not govern the steel area requirements for these small pipe except, perhaps, in Class IV and higher strength classes.

Comparison of Eq. 7 with a similar equation for welded smooth wire fabric (11) indicates an increase of 10 to 25 percent in 0.01-in. crack strength for welded deformed wire fabric.

As a further limitation, Eq. 7 will apply only where the reinforcing steel stress at 0.01-in. crack load (as calculated by Eq. 1) is still below the yield point of the reinforcing steel. Thus, for lower strength reinforcement and pipe designs with low steel percentages (i.e., Class II), the 0.01-in. crack load calculated using Eq. 7 should also be checked against the following equation (derived by rearrangement of Eq. 1 and substituting  $f_{s,01} = f_y$ ):

$$\max DL_{.01} = \frac{72 A_{s1} d_1 f_y}{D_i^2} - 9 \frac{W}{D_i} \quad (8)$$

#### Ultimate Strength—Types of Failure

Ultimate failure of reinforced concrete pipe may be by flexure with tensile failure of the reinforcing steel, by diagonal tension failure of the concrete, or by a combination of excessive yielding of the reinforcing steel and diagonal or radial tension ("slabbing") failure in the concrete. Pipe with relatively low percentages of cold-drawn reinforcing will fail by rupture of the reinforcing steel after substantial yielding has occurred at crown, invert, and springing sections of the pipe (flexural failure). Pipe with somewhat larger amounts of cold-drawn reinforcing will fail suddenly by formation of an inclined diagonal crack in the concrete at invert or crown or by sudden "slabbing" off of concrete cover at this location.

#### Flexural Strength

During ultimate flexural behavior of pipe, cold-drawn wire reinforcing, whether smooth or deformed, has sufficient ductility to allow plastic rotation of sections at the crown and invert. The steel initially yields at these locations under a load well below

the ultimate load, so that favorable redistribution of bending moments can occur. As the crown and invert sections undergo yielding at a low rate of increase in resisting moment, the resisting moment increases more rapidly at the springings where the steel is less highly stressed. Fortunately, the ductility of cold-drawn wire is such that before final ultimate flexural failure occurs, both the inside steel and the outside steel reach their full ultimate strength, or a very high percentage of this strength (3). When this occurs, the pipe has developed the maximum possible flexural load-carrying ability of its constituent materials.

#### Ultimate Flexural Strength Equation

On the basis of the so-called plastic hinge theory, Heger (3) developed formulas for ultimate flexural strength of pipe with cold-drawn wire or fabric reinforcing under 3-edge bearing load. Previous test results on pipe with conventional welded wire fabric indicate good correlation between tests and this ultimate flexural theory. These formulas for ultimate flexural strength are summarized in the following.

For pipe with wall thickness  $5\frac{1}{2}$  in. or larger having two lines of welded (deformed or smooth) wire fabric reinforcing:

$$DL_u = \frac{87.5 c f_{su1} A_{s1} (d_1 - 0.5 a)}{D_i^2} - 6 \frac{W}{D_i} \quad (9)$$

where

$$a = 0.1 \frac{f_{su1} A_{s1}}{f'_c} \quad (10)$$

$$c = 0.57 \frac{(1 + f_{su2} A_{s2} d_2)}{f_{su1} A_{s1} d_1} \quad (11)$$

For pipe with wall thickness less than  $5\frac{1}{2}$  in. having two lines of welded (deformed or smooth) wire fabric reinforcing:

$$DL_u = \frac{91.7 c f_{su1} A_{s1} (d_1 + 0.80 - 0.88 a')}{D_i^2} - 6 \frac{W}{D_i} \quad (12)$$

where

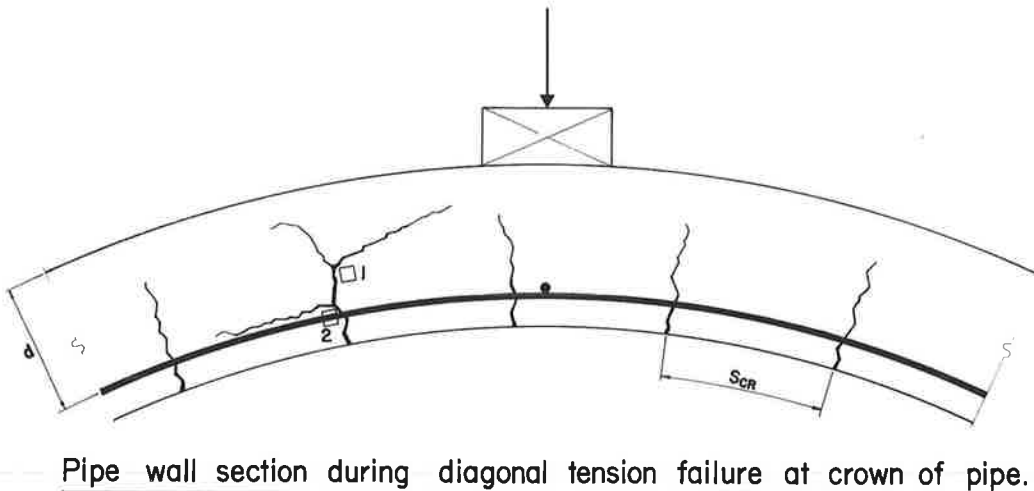
$$a' = 0.175 \frac{f_{su1} A_{s1}}{f'_c} \leq 0.8 \text{ in.} \quad (13)$$

If  $a' > 0.8$  in., use Eqs. 9 and 10.

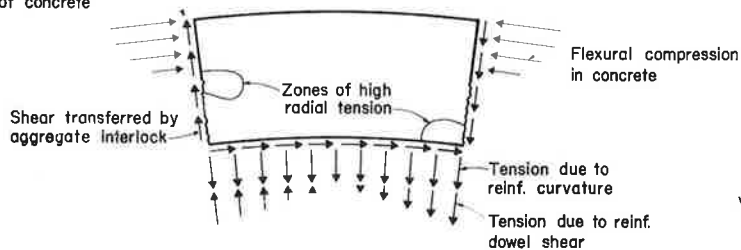
#### Diagonal Tension Strength

For pipes without stirrups and with cold-drawn wire or wire fabric reinforcing, failure by diagonal tension (sometimes called shear failure) is characterized by the sudden formation of a crack extending from just above the inside layer of steel diagonally toward the support point at the outside of the pipe at the bottom, or toward the load point at the top. Alternately, it may be characterized by the sudden formation of a circumferential crack just above the inside steel near the crown or invert of the pipe which allows slabbing off of concrete cover and straightening of inside reinforcing. Slabbing, either by itself or in conjunction with diagonal cracking, is more likely to occur with a fabric having widely spaced longitudinals (i. e., greater than 8 in.). Slabbing is even more predominant when hot-rolled reinforcing is used, because such reinforcing usually is used with widely spaced longitudinals and may also undergo very large plastic strains

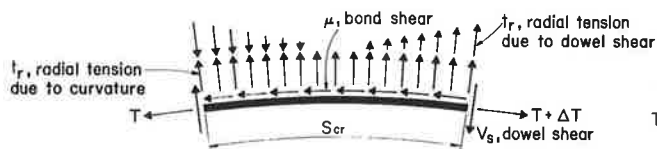
at the crown and invert. Test results indicate that slabbing and diagonal cracking are probably manifestations of the same phenomena in pipe reinforced with cold-drawn wire or wire fabric reinforcing. In the ensuing discussion, the term diagonal tension failure will denote either of the above modes of failure.



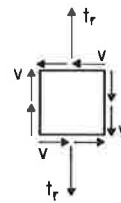
Shear carried in compression zone of concrete



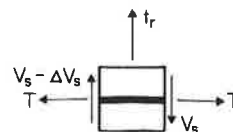
#### Forces on concrete tooth



#### Forces on reinforcing between two cracks

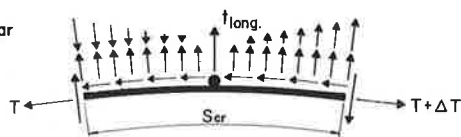


FREE BODY 1



FREE BODY 2

#### Combined stresses on concrete element



#### Forces on reinforcing with longitudinal bar between two cracks

Figure 9. Mechanism of diagonal tension failure.

### Mechanism of Diagonal Tension Failure

Although structural design to resist diagonal tension failure in general concrete building construction is a routine task, the precise mechanism by which such failure occurs has not been determined. However, one theory which has recently been advanced by Kani (12) seems to offer a plausible explanation for the mechanism of diagonal tension failure. This theory can best be explained by examining the following sequence of events which occur when a pipe is subjected to 3-edge bearing load:

1. Cracks form in regions of high flexural stress at the crown and invert. These cracks extend from the inner surface in a radial direction.
2. The variation in bending moment requires a corresponding variation in the tensile force in the inner reinforcing. This is accomplished through bond (i. e., horizontal shear) between the steel and the surrounding concrete.
3. This horizontal shear force is maximum in the zone of flexural cracks at the crown and invert. It acts to bend the block or "tooth" of concrete which extends down from the neutral axis between two flexural cracks (Fig. 9).

It is evident from the foregoing qualitative discussion that diagonal tension failure is indeed a complex phenomenon. However, physical reasoning based on several somewhat different approaches leads to the same general conclusions about the major variables which influence diagonal tension behavior.

### Theories of Diagonal Tension Failure

Both the "combined stress" theory of Viest and ACI Committee 326 (13) and the "tooth" theory of Kani (12) indicate that the shear strength of concrete members subject to flexural cracks in the shear zone is influenced by the following parameters:

$$v = C_3 \sqrt[n]{f'_c} + C_4 p \frac{Vd}{M} \quad (14)$$

In addition, the tooth theory indicates that the depth of section is also an influencing parameter. The following expression is similar in form to Eq. 14, but takes depth of section into account:

$$v = \frac{C_5 \sqrt[n]{f'_c}}{(d + C_7)} + C_6 p \frac{Vd}{M} \quad (15)$$

When Eq. 15 is extended to include radial tension due to the effect of steel curvature and is applied to pipe under 3-edge bearing load which have the dimensional proportions specified in ASTM C 76, the following relationship of significant parameters for ultimate diagonal tension D-Load strength is obtained:

$$DL_u = \frac{C_8 d_1 \sqrt[n]{f'_c}}{(d_1 + C_{10}) D_i} + \frac{C_9 A_{s1} d}{D_i^2} - 11 \frac{W}{D_i} \quad (16)$$

### Evaluation of Constants

Test data from the U. S. Steel test program and from the other two independent programs are plotted in terms of the above parameters with  $n = 3$  and  $C_{10} = 11$  (Fig. 10). The constants  $C_8$ ,  $C_9$ ,  $C_{10}$  and  $n$  were determined by a statistical evaluation of the test data. A computer program was developed to carry out the large number of calculations required for this analysis. A discussion of the statistical evaluation follows.

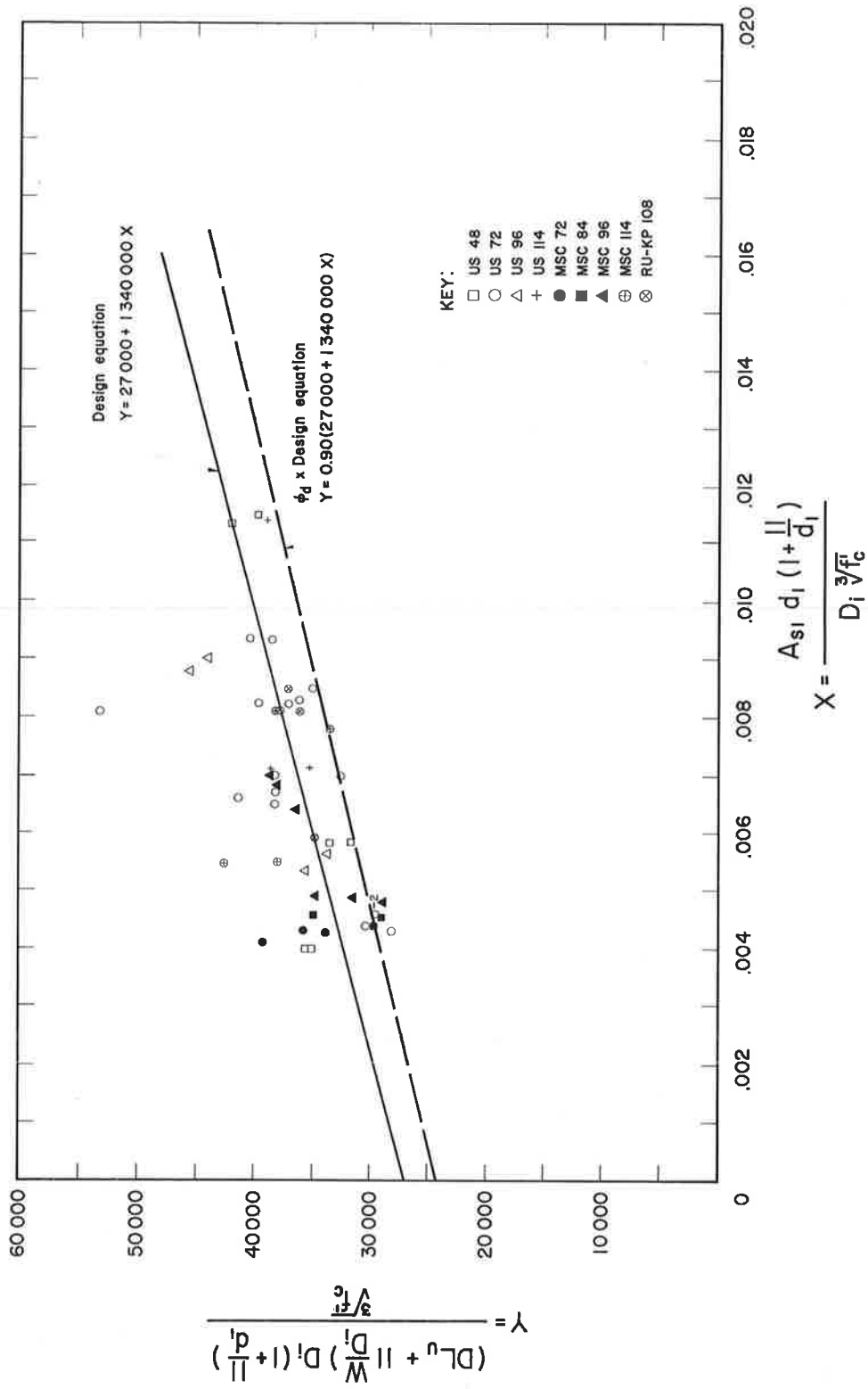


Figure 10. Development of diagonal tension D-Load strength equation.

1. Preliminary review of data indicated that two trial values of  $n$  might be used,  $n = 2$  and  $n = 3$ . A separate analysis for each of these values of  $n$  was made.
2. Values of  $C_8$  and  $C_9$  were calculated by least squares analysis for a large number of trial values of  $C_{10}$ . Trial values of  $C_{10}$  extend from 0 to 400.
3. The standard deviation was calculated for each trial value of  $C_{10}$  and the associated least squares values of  $C_8$  and  $C_9$ .
4. The trial value of  $C_{10}$  and associated least squares values of  $C_8$  and  $C_9$  which give the lowest standard deviation were deemed the best fit of the data to Eq. 16.
5. The existence of a minimum standard deviation was confirmed by checking for zero slope of the equation for standard deviation in terms of  $C_8$ ,  $C_9$ , and  $C_{10}$ . The three partial differentials of the standard deviation with respect to  $C_8$ ,  $C_9$ , and  $C_{10}$  were set equal to zero. Solution of these equations for various trial values of  $C_{10}$  by computer indicated the values of  $C_8$ ,  $C_9$ , and  $C_{10}$  which produced simultaneous zero values for all three partial differential equations. It was then verified that these values actually do produce the best correlation of test data with the variables selected for analysis. This process indicated that the constants which gave the lowest standard deviation were:  $n = 3$ ,  $C_8 = 26,200$ ,  $C_9 = 1,340,000$ ,  $C_{10} = 10.3$ . However, for ease in calculation, these values were rounded off, with no significant loss in correlation, as follows:  $n = 3$ ,  $C_8 = 27,000$ ,  $C_9 = 1,340,000$ ,  $C_{10} = 11$ .

#### Comparison With Constants in Previous Diagonal Tension Equation

Only a slightly higher standard deviation was obtained if  $n = 2$ , rather than  $n = 3$ , was used. In this case  $C_8 = 675$ ,  $C_9 = 1,490,000$ ,  $C_{10} = 12.5$ . When  $n = 2$ , Eq. 16 is similar in form to Heger's (3) Eq. 39 (with units of  $D_i$  changed to inches):

$$DL_u = \frac{450 d_i \sqrt{f'_c}}{D_i} + \frac{1,120,000 A_{s1} d_i}{D_i^2} - 11 \frac{W}{D_i} \quad (39)$$

The terms  $C_8/(d + C_{10})$  and  $C_9$  in Eq. 16 are similar to the first two constants in Eq. 39.

The value  $C_8/(d + C_{10})$  depends on pipe wall thickness. Thickness is a new parameter which was not taken into account in Eq. 39. However, the constants in Eq. 39 were derived from tests carried out, for the most part, on Wall B 72-in. diameter pipe. For this pipe size, the quantity  $C_8/(d + C_{10}) = 370$  for the present investigation. This may be compared with the constant 450 in Eq. 39 for the previous investigation. The new tests indicate a lower first term constant than previously obtained. The higher second term constant of the new tests makes up only a small part of the difference in  $DL_u$  due to the first term in a comparison of the two diagonal tension equations.

#### Influence of Fabric Longitudinals

This additional discrepancy between the present and previous diagonal tension equations can be explained by examining the details of the reinforcement used in the supporting test programs. In the earlier tests, conventional wire fabric was used, where the circumferential spacing of longitudinals was 8 in. or less. In the recent test program, deformed wire fabric was used, and the longitudinals were spaced at either 12 or 16 in. The diagonal tension strengths observed in the former program were somewhat higher than those observed in the present tests—the difference in  $DL_u$  being 200 to 300 lb per sq ft. This finding, together with similar results observed in some recent tests by others, indicates that the spacing of longitudinal wires may influence the diagonal tension strength of pipe. This behavior was not previously considered as a parameter which influences diagonal tension strength.

Physical reasoning indicates that it is plausible to obtain an increase in diagonal tension strength if longitudinal wires are spaced sufficiently close together. Longitudinal wires can help to distribute both the radial tension force due to reinforcing curvature and the dowel shear (Fig. 9) which exists at flexural cracks. This spreads the tension in the concrete "teeth" more evenly and lowers the maximum concrete tensile stress in a tooth. For this to be possible, spacing of longitudinal wires must be nearly equal

to, or less than, flexural crack spacing. This indicates a 6- to 8-in. maximum spacing of longitudinal wires for effective distribution of radial stress.

A quantitative evaluation of the effect of closely spaced longitudinals acting as partially effective shear reinforcing in concrete pipe must be developed empirically from comparative test data. At present virtually no data of this nature are available. The test programs reported herein were not designed to investigate these newly found effects.

#### Influence of Wire Deformations

A portion of the lower diagonal tension strength observed in the current test data possibly may be caused by effects due to closer spacing of flexural cracks with deformed wire. According to the tooth theory previously described, crack spacing determines the depth of tooth for resistance to tooth bending. Shorter teeth may be weaker, although some of the strength reduction due to loss of tooth depth is very likely made up by increases in dowel and aggregate interlock shear forces due to narrower width of crack with deformed wire. At present, neither the tooth theory nor the available test data are sufficient to indicate whether the existence of deformations on the wire contributes to lower diagonal tension strength observed in this test program.

#### Ultimate Diagonal Tension Strength Equation

Further modifications of Eq. 16 must be made for production pipe with standard tongue and lip ends. These pipe do not have the full wall thickness over their entire nominal length. Consequently, a factor must be included for the ratio of effective length of full thickness barrel to nominal length of the pipe section.

The final design equation for ultimate diagonal tension strength is as follows:

$$DL_u = \left[ \frac{27,000 d_1 \sqrt{f'_c}}{D_i (d_1 + 11)} + \frac{1,340,000 d_1 A_{s1}}{D_i^2} + C_L N_L \right] \frac{L_e}{L_n} \phi_d - 11 \frac{W}{D_i} \quad (17)$$

where  $C_L$  is a constant which depends on the spacing of longitudinals in welded wire fabric. Tentatively (subject to confirmation by more test data),  $C_L$  equal to 200 lb per sq ft for inner cage welded wire fabric having longitudinals spaced at 8 in. or less seems reasonable. For greater spacing of fabric longitudinals,  $C_L = 0$ .  $N_L$  is the number of wraps of fabric used for the inner line of reinforcing at crown and invert and must be limited by practical considerations of steel placement.

#### Interaction Between Flexural Ultimate Strength and Diagonal Tension Ultimate Strength

When the properties of a pipe are such that Eq. 9 for ultimate flexural strength and Eq. 17 for ultimate diagonal tension strength indicate nearly the same  $DL_u$ , the expected failure mode may be termed flexural-diagonal tension. For this situation, conditions associated with the approach of flexural ultimate strength (i.e., large steel strains) may influence the ultimate diagonal tension strength, and vice versa. A review of test results from the U.S. Steel test program indicates that the 48-in. diameter Class III and the 96-in. diameter Class II pipe were in this classification. Although few test specimens were involved, they all showed slightly lower ultimate strengths than indicated by the semi-empirical ultimate strength equations developed above. However, it does not appear necessary to incorporate any provision into the two independent ultimate strength equations for this possible interaction effect if the precautions suggested in the design procedure presented herein are followed.

### CORRELATION OF THEORY AND TESTS

#### 0.01-In. Crack Strength

Values for  $DL_{01}$  are calculated for all test pipe using Eq. 7. These calculations are made using actual measured values for wall thickness, concrete cover thickness,



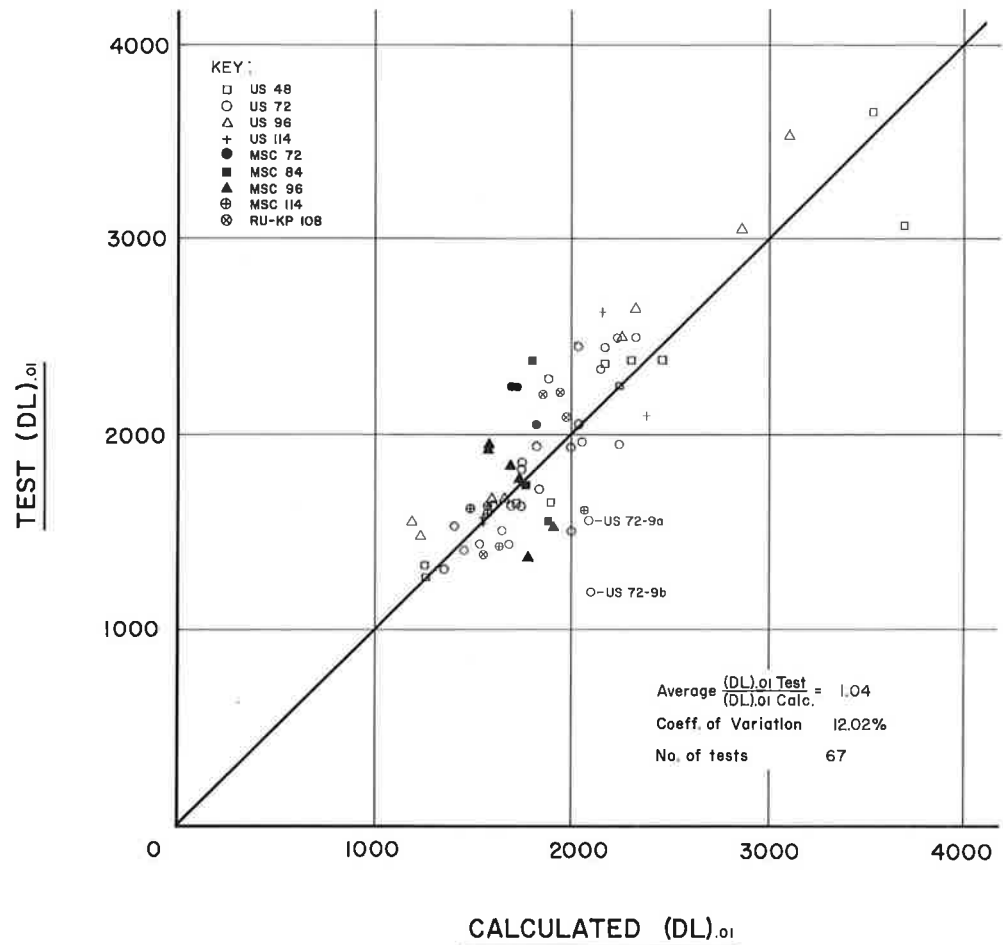


Figure 11. Comparison of test and calculated 0.01-in. crack strengths.

steel area, and concrete strength for each test specimen. Test and calculated D-Loads are compared in Figure 11. For 47 of the 49 test specimens of the U.S. Steel test program, the average  $DL_{0.1} \text{ test} / DL_{0.1} \text{ calc} = 1.03$ . Specimens US 72-9a and 9b are not included because of obviously erratic results. The coefficient of variation is 10.5 percent. For 20 of the 21 test specimens in the two other test programs, the average  $DL_{0.1} \text{ test} / DL_{0.1} \text{ calc} = 1.06$ . Specimen MS 114-1b is not included because of inadequate cover over reinforcing at a critical location. The coefficient of variation is 15.6 percent. The combined average  $DL_{0.1} \text{ test} / DL_{0.1} \text{ calc}$  for all of the test specimens is 1.04. The combined coefficient of variation is 12.0 percent.

Note that only test results from the U.S. Steel program were used to develop the semi-empirical constants in Eq. 7. Therefore, the comparison of  $DL_{0.1} \text{ test}$  to  $DL_{0.1} \text{ calc}$  for the other test programs provides an independent check of the validity of Eq. 7.

#### Ultimate Strength

Equation 9 or 12 is used to calculate values for ultimate flexural strength  $DL_u$  for all test specimens which failed in flexure. Equation 17 is used to calculate values for ultimate diagonal tension strength  $DL_u$  for all test specimens which failed in diagonal

tension. The term  $C_{LN_L}$  in Eq. 17 is assumed to be 200 for each inner wrap of reinforcing at the crown and invert, with longitudinals at 8-in. maximum spacing. For two inner wraps with longitudinals spaced at 16 in. or less,  $C_{LN_L}$  is assumed to be 200. For single inner wraps with longitudinals spaced greater than 8 in.,  $C_{LN_L} = 0$ . All calculations are made using actual measured values for wall thickness, concrete cover thickness, steel area, concrete strength, and steel strength.

Test and calculated D-Loads are compared in Figure 12 for flexural ultimate strength and in Figure 13 for diagonal tension ultimate strength. For those test specimens having nearly the same calculated D-Load ultimate for both flexural failure and diagonal tension failure, the ratio  $DL_u \text{ test}/DL_u \text{ calc}$  used is that which corresponds to the observed failure mode. For the nine test specimens which failed in flexure, the average  $DL_u \text{ test}/DL_u \text{ calc} = 0.98$ . Coefficient of variation is 6.53 percent. For the 57 test specimens which failed in diagonal tension, the average  $DL_u \text{ test}/DL_u \text{ calc} = 1.01$ . Coefficient of variation is 11.8 percent.

Note that the results of other programs, as well as the U.S. Steel program results, were all used for the determination of constants in the diagonal tension equation, Eq. 17. This was necessary to take advantage of the maximum amount of available data over the full range of pipe sizes and strengths of interest for C 76 designs.

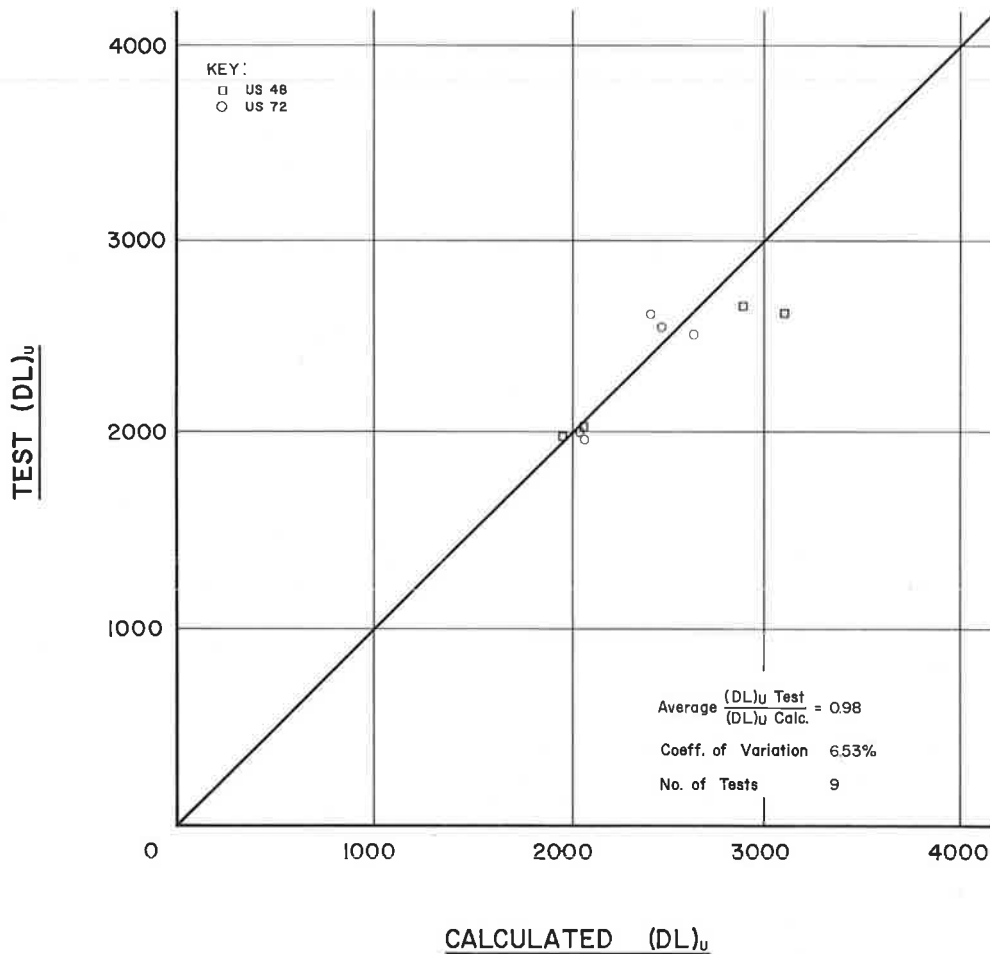


Figure 12. Comparison of test and calculated ultimate flexural strengths.

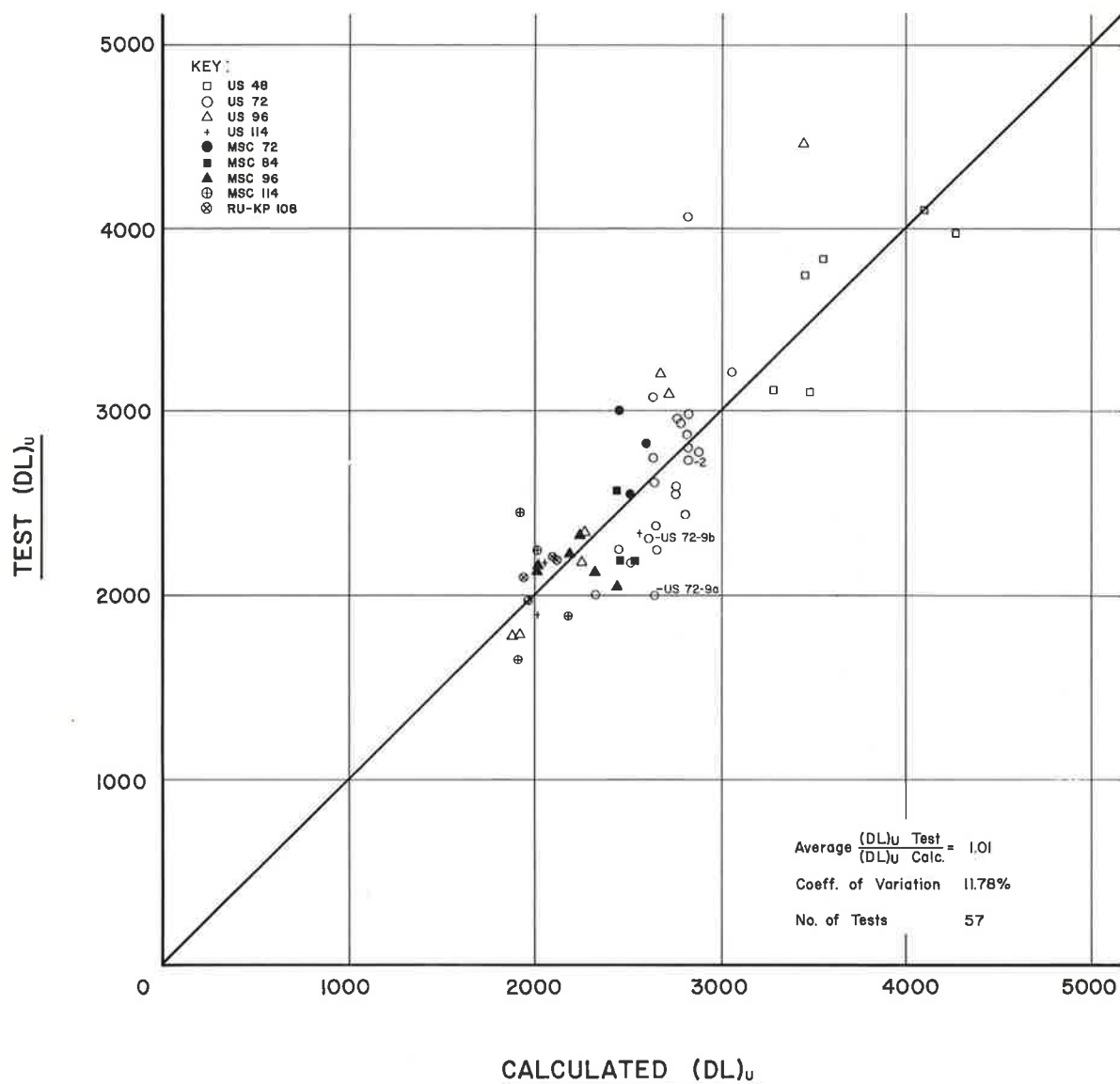


Figure 13. Comparison of test and calculated ultimate diagonal tension strengths.

#### Calculated Steel Stresses

Equation 1 is used to calculate the maximum elastic theory steel stress in the inner cage reinforcing for the test 0.01-in. crack load and for the test ultimate load. Results are given in Table 5 for the U.S. Steel program and in Table 6 for the other test programs. Steel yield strength (0.2 percent offset) and ultimate strength values are also indicated in the tables for each test specimen.

For the 0.01-in. crack load, the maximum steel stress is less than the steel yield strength for nearly all test specimens. The magnitude of this stress varies greatly depending on the percentage of steel reinforcing used. In general, the stress level at measured 0.01-in. crack load decreases with increasing steel percentage.

TABLE 5  
CALCULATED STEEL STRESSES AT 0.01-IN. CRACK AND  
ULTIMATE LOADS—U. S. STEEL TEST PROGRAM

Pipe Mark	Steel Properties		Calculated Max. Steel Stress Elastic Theory	
	Yield	Ultimate	at	at
	Str. by Test <sup>a</sup> ksi	Str. by Test ksi	(DL) <sub>.01</sub> ksi	(DL) <sub>U</sub> ksi
US 48-1a	91.2	98.3	88.2	131.0*
US 48-1b	91.2	98.3	90.5	133.0*
US 48-2a	99.4	96.2	75.4	115.0*
US 48-2b	99.4	96.2	79.6	123.0*
US 48-3a	87.7	92.6	57.3	73.7
US 48-3b	87.7	92.6	57.3	74.1
US 48-4a	81.1	82.9	34.5	44.5
US 48-4b	81.1	82.9	42.5	47.3
US 48-5a	80.8	90.5	70.8	117.0*
US 48-5b	80.8	90.5	76.6	118.0*
US 72-1a	85.8 <sup>1</sup>	101.0	55.0	86.1*
US 72-1b	85.0 <sup>1</sup>	101.0	57.1	81.0
US 72-1c	84.6 <sup>1</sup>	101.0	50.2	79.4
US 72-2a	90.9 <sup>1</sup>	95.3	56.2	87.6
US 72-2b	90.9 <sup>1</sup>	94.5	57.9	80.2
US 72-2c	90.9 <sup>1</sup>	94.0	45.6	83.6
US 72-3a	70.5 <sup>1</sup>	78.4	70.5	88.7*
US 72-3b	67.6 <sup>1</sup>	75.1	59.2	94.4*
US 72-3c	68.0 <sup>1</sup>	75.6	65.6	99.6*
US 72-4a	86.5	89.2	85.5	107.0*
US 72-4b	86.5	89.2	77.7	110.0*
US 72-5a	85.8	95.3	57.0	77.3
US 72-5b	85.8	95.3	59.4	85.0
US 72-6a	82.5	90.8	42.1	59.9
US 72-6b	82.5	90.8	47.7	56.9
US 72-7a	77.4	85.9	57.8	65.2
US 72-7b	77.4	85.9	56.2	62.3
US 72-8a	77.4	85.9	43.2	51.7
US 72-8b	77.4	85.9	53.0	58.7
US 72-9a	77.4	85.9	37.5	46.3
US 72-9b	77.4	85.9	30.6	54.3
US 72-10a	74.4	85.4	52.7	62.3
US 72-10b	74.4	85.4	55.9	89.2*
US 72-12a	78.4	87.3	64.2	94.0*
US 72-12b	78.4	87.3	69.8	91.5*
US 72-13a	81.5	91.4	61.5	77.3
US 72-13b	81.5	91.4	50.9	62.9
US 96-1a	85.1	95.4	85.9*	96.1*
US 96-1b	85.1	95.4	78.0	91.1*
US 96-2a	80.6	90.4	62.7	78.7
US 96-2b	80.6	90.4	64.3	86.3*
US 96-3a	79.2	81.9	56.1	70.0
US 96-3b	79.2	81.9	57.2	65.7
US 96-4a	—	—	44.4	55.2
US 96-5a	74.5	83.7	—**	—**
US 114-1a	82.5	90.8	55.7	65.3
US 114-1b	82.5	90.8	54.5	72.8
US 114-2a	80.8	88.4	45.7	49.6
US 114-3a	80.8	88.4	50.4	—**

## Notes:

1. Assumed yield strength equals .9 ultimate strength  
2. Yield strength at 0.2% offset

\* Exceeds yield strength -  
Elastic Theory not applicable

\*\*Stirrups

TABLE 6  
CALCULATED STEEL STRESSES AT 0.01-IN. CRACK AND ULTIMATE LOADS—  
OTHER TEST PROGRAMS

Pipe Mark	Steel Properties		Calculated Max. Steel Stress Elastic Theory	
	Yield	Ultimate	at	at
	Str. by Test <sup>2</sup> ksi	Str. by Test ksi	(DL) <sub>.01</sub> ksi	(DL) <sub>U</sub> ksi
RU-KP 108-1a	85.2 <sup>1</sup>	94.7	58.7	58.7
RU-KP 108-1b	85.2 <sup>1</sup>	94.7	59.8	59.8
RU-KP 108-2	85.2 <sup>1</sup>	94.7	56.2	76.1
RU-KP 108-3	81.0 <sup>2</sup>	90.0	54.8	77.8
MS 72-1a	87.2	92.1	78.8	106.0*
MS 72-1b	87.2	92.1	94.0*	122.0*
MS 72-1c	87.2	92.1	89.7*	100.0*
MS 84-1a	80.7	92.2	70.1	85.7*
MS 84-1b	80.7	92.2	61.0	81.6*
MS 84-1c	80.7	92.2	90.6*	97.4*
MS 96-1a	81.5	91.4	73.3	89.8*
MS 96-1b	81.5	91.4	59.2	76.2
MS 96-1c	81.5	91.4	55.3	80.2
MS 96-2a	74.7	83.0	59.0	71.5
MS 96-2b	74.7	83.0	65.7	70.7
MS 96-2c	74.7	83.0	63.9	70.8
MS 114-1a	82.5	90.8	67.4	97.0*
MS 114-1b	82.5	90.8	56.5	65.0
MS 114-1c	82.5	90.8	65.7	86.2*
MS 114-2a	87.3	97.0	53.4	54.9
MS 114-2b	87.3	97.0	47.3	62.4

Notes:

1. Assumed yield strength equals .9 ultimate strength

2. Yield strength at 0.2% offset

\* Exceeds yield strength—

Elastic Theory not applicable

For the ultimate load, the maximum steel stress, as calculated by the elastic theory, is valid only where failure is by diagonal tension with no yield in the inner reinforcing steel. These calculated stresses have no meaning for specimens failing in flexure, or specimens failing in diagonal tension after yield of the steel. Indeed, calculated stresses substantially exceed the ultimate strength of the reinforcing for specimens failing in the flexural mode or failing nearly balanced between diagonal tension and flexural. This is another illustration of the effect of plastic rotation and redistribution of moment during flexural ultimate failure of pipe with cold-drawn wire reinforcing.

#### DESIGN OF ASTM C 76 CONCRETE PIPE WITH WELDED DEFORMED WIRE FABRIC REINFORCING

##### Design Procedure

Equations were developed in the preceding sections for calculation of the 0.01-in. crack strength and ultimate strength of circular concrete pipe with welded deformed wire fabric under 3-edge bearing load. When rearranged in a more convenient form, these equations provide a basis for the design of such pipe to meet performance criteria established in ASTM C 76. The following stepwise procedure is suggested for design of C 76 pipe with welded deformed wire fabric reinforcing.

1. For a particular 3-edge bearing strength requirement and internal diameter, select wall thickness, steel location, arrangement, and tolerances in accordance with the requirements of ASTM C 76.

2. ASTM specifications for welded deformed wire set ultimate tensile strength of steel at 80,000 psi, minimum.

3. Select design variability factors  $\phi_f$ ,  $\phi_d$ , and  $\phi_{.01}$ . These factors allow for the variability between design theory and test results for ultimate load and 0.01-in. crack load, respectively. For design based on ultimate flexural strength,  $\phi_f = 0.95$  is suggested on the basis of existing test results. Somewhat greater variability must be expected in ultimate diagonal tension strength and 0.01-in. crack strength test results; consequently,  $\phi_d$  and  $\phi_{.01} = 0.90$  are suggested for design to meet these strength requirements.

4. Determine a pipe fabrication and materials variability factor,  $\phi_x$ . This factor should reflect quality control standards in the pipe industry or at a particular pipe fabricator. Depending on these quality control standards, the value of  $\phi_x$  might range from 0.85 to 0.92 for pipe having complete circular cages for inner and outer lines of reinforcing. For pipe with a single elliptical cage,  $\phi_x$  should probably be less than for two-cage pipe to allow for the greater possibility of misplacement of steel.

5. Determine a trial area for inner cage reinforcing based on ultimate flexural strength as follows: (a) For pipe with wall thickness  $5\frac{1}{2}$  in. or larger with outer cage area at springings three-fourths of inner cage area at crown and invert,

- (1) estimate trial values of  $f'_c$  and  $A_{s1}$  and calculate:

$$a = 0.1 \frac{f_{su}}{f'_c} A_{s1} \quad (18)$$

- (2) Determine required steel area:

$$A_{s1} = \frac{0.0115 D_i^2 \left( DL_u + 6 \frac{W}{D_i} \right)}{f_{su} (d - 0.5a) \phi_f \phi_x} \quad (19)$$

(b) For pipe with wall thickness less than  $5\frac{1}{2}$  in. with outer cage area three-fourths of inner cage area,

- (1) estimate trial values for  $f'_c$  and  $A_{s1}$  and calculate:

$$a' = 0.175 \frac{f_{su}}{f'_c} A_{s1} \quad (20)$$

- (2) Determine required steel area:

$$A_{s1} = \frac{0.011 D_i^2 \left( DL_u + 6 \frac{W}{D_i} \right)}{f_{su} (d + 0.8 - 0.9a') \phi_f \phi_x} \quad (21)$$

6. Select spacing of circumferential wires.

7. Also determine trial area for inner cage reinforcing based on 0.01-in. crack strength as follows: (a) For 48-in. and larger diameter pipe with outer cage area at springings at least three-fourths of inner cage areas at crown and invert:

$$A_{s1} = \frac{3.1 \times 10^{-7} D_i^2 \sqrt{A_{cs}} \left[ \left( DL_{.01} + 9 \frac{W}{D_i} \right) \frac{1}{\phi_{.01}} - \frac{144 h \sqrt{f'_c}}{D_i} \right]}{d \phi_x} \quad (22)$$

Check:

$$A_{s1} = \frac{0.014 D_i^2 \left( DL_{.01} + 9 \frac{W}{D_i} \right)}{df_y \phi_x} \quad (23)$$

(b) For pipe smaller than 48-in. diameter, no 0.01-in. crack design formula is available. Equations 22 and 23 probably give increasingly more conservative results as pipe diameter decreases.

8. Select the larger area,  $A_{s1}$ , from Steps 5 and 7 above.
9. Check for adequate ultimate diagonal tension strength:

$$DL_u = \left[ \frac{27,000 d \sqrt{f_c}}{D_i (d + 11)} + \frac{1.34 \times 10^6 d A_{s1} \phi_x}{D_i^2} + C_{LNL} \right] \frac{L_e}{L_n} \phi_d - 11 \frac{W}{D_i} \quad (24)$$

If the calculated  $DL_u$  is less than the required  $DL_u$ , the following means are available to increase  $DL_u$  calculated:

(a) (tentative) Welded deformed wire fabric with longitudinals spaced at 8 in. or less for inner cage reinforcing: Magnitude of  $DL_u$  increase available is approximately 180 lb per sq ft.

(b) (tentative) A double layer of inner cage welded wire fabric with longitudinals spaced at 8 in. or less in each layer, over a circumferential length at least equal to  $0.6 D_i$  at crown and invert regions: Place circumferentials directly over each other to maintain sufficient space between adjacent circumferentials for adequate concrete placement. Magnitude of  $DL_u$  increase available is approximately 360 lb per sq ft.

(c) Higher concrete strength: Practical upper limit is from about 5500 to 6000 psi depending on the quality of available aggregates. The approximate increase in  $DL_u$  with concrete strength increased from 5000 to 6000 psi is 140 lb per sq ft for 48-in. diameter, Wall B, and 90 lb per sq ft for 120-in. diameter, Wall B.

(d) Increase circumferential steel area: The approximate magnitude of increase in  $DL_u$  for each 0.001 sq in. per ft-in. of  $A_{s1}/D_i$  ratio is 80 lb per sq ft (Wall B pipe).

(e) Provide stirrup reinforcing extending in a radial direction between inner and outer cages at crown and invert: To be fully effective, stirrups must be spaced circumferentially at no more than about three-fourths of the effective depth of section, and longitudinally so as to tie each circumferential wire. They must extend circumferentially over a region equal to about  $0.6 D_i$  at crown and invert, and must be adequately anchored to inner and outer circumferential reinforcing. A tentative design procedure for fully effective stirrups is presented under Step 12 of this design section. Partially effective stirrup reinforcing may also be used to obtain relatively small increases in  $DL_u$ . Most partially effective stirrup systems meet all the requirements for fully effective stirrups, except they may tie only every second or third circumferential wire. Staggered patterns may be used to increase the effectiveness of partially effective stirrup systems.

10. If the diagonal tension  $DL_u$  (Eq. 24) is between 1.00 and 1.10 times the required  $DL_u$ , increase  $A_{s1}$  required for flexure (i.e., Eq. 19 or 21) to  $A'_{s1}$  as follows:

$$A'_{s1} = A_{s1} \left[ 1 + \frac{1.10 DL_{u-reqd} - DL_{u-Eq. 24}}{2 DL_{u-reqd}} \right] \quad (25)$$

(For  $A_{s1}$  refer to Eq. 19 or 21.)

11. Determine outer cage steel area:

$$A_{s2} \geq 0.75 A_{s1} \quad (26)$$

where  $A_{s1}$  = the larger of the steel areas required for flexural ultimate strength or 0.01-in. crack strength.

12. Where fully effective stirrup reinforcing is used to obtain required diagonal tension ultimate strength, select stirrup area (tentative to be confirmed by tests):

$$A_v = \frac{0.10 D_i s}{f_y d \phi_x} \left[ \left( DL_u + 11 \frac{W}{D_i} \right) \frac{1}{\phi_d} - \left( \frac{27,000 d \sqrt[3]{f'_c}}{D_i (d + 11)} + \frac{1.34 \times 10^6 d A_{s1} \phi_x}{D_i^2} \right) \frac{L_e}{L_n} \phi_d \right] \quad (27)$$

but not less than

$$A_v = \frac{0.03 D_i s}{f_y d \phi_x} \left( DL_u + 11 \frac{W}{D_i} \right) \frac{1}{\phi_d} \quad (28)$$

where

$$s_{max} = 0.75 d \quad (29)$$

Note that stirrups must be securely anchored at both ends by substantial hooks, welds, or other means sufficient to develop the yield strength of the stirrup. Stirrup placement should extend over a circumferential length equal to  $0.6 D_i$  at crown and invert.

#### Limitations of Design Procedure

The preceding design procedure should be valid under the following conditions:

1. Reinforcing: Welded deformed wire fabric with bond characteristics equivalent to U.S. Steel's deformed wire. No limitation on spacing of longitudinals except where noted in design procedure. Spacing of circumferential reinforcing is 2 in. where diagonal tension without stirrups governs; 2 to 3 in. where 0.01-in. crack governs; and 2 to 4 in. where ultimate flexure governs.
2. Pipe strength classes: II to V in ASTM C 76. This means relative steel area  $A_{s1}/D_i$  may vary from about 0.003 to 0.015 sq in. per ft-in.
3. Pipe diameter: 48 to 120 in.
4. Pipe wall thickness: C 76 Wall A to C.
5. Concrete strength: 4000 to 6000 psi.
6. Cover over reinforcing: 1 in.  $\pm \frac{3}{8}$  in.
7. Method of manufacture must produce bond of concrete to reinforcing equivalent to or better than the cast process for the same strength concrete.

Designs based on this procedure, but outside the range of variables listed above, must be considered tentative until confirmed by proper testing. Furthermore, application of this design procedure to particular design problems should be carried out with the following understanding of characteristics inherent in the development of the method.

1. Constants in the equations for 0.01-in. crack strength and ultimate diagonal tension strength are semi-empirically derived from analysis of the test data described in this report. These constants may require modification for conditions markedly different from those existing in the test program. For example, all test pipe were made by the cast process. Any manufacturing process which causes inferior, or superior, bond between concrete and reinforcing steel, or other significant characteristics of concrete not reflected in the concrete compressive strength,  $f'_c$ , may produce pipe whose structural behavior is not accurately reflected by the design equations.

2. Because of the brittle behavior of concrete in tension, substantial variability must be expected in 0.01-in. crack strength and ultimate diagonal tension strength. Inherent variability exists in the nature of basic materials which constitute concrete, in the assembly of these materials to produce concrete, and in the many factors in the pipe fabrication process which may affect the final structure. The variability allowances which are suggested above are believed to be adequate for successful design in most instances.



However, of necessity, sampling for the test program was limited to three manufacturers at three different locations. Greater variability may occur from time to time due to local conditions markedly different from those which existed for the test program.

3. In order to meet the performance criteria of C 76 with significant economy over current tabular designs, all designs must have low factors of safety relative to test requirements. In light of this and the preceding comments, new designs based on the procedure suggested herein should be confirmed by a few check tests to insure that no problems related to the particular local material and fabrication conditions occur.

4. Certain parts of the design procedure given are labeled "tentative." These tentative provisions in the design procedure are developed from incomplete, or inadequate, test data. Further tests are required to fully evaluate these tentative aspects of the design procedure.

### CONCLUSIONS

A review of steel area requirements, as obtained by both C 76 tabular designs and the design equations developed herein, indicates that welded deformed wire fabric may provide substantial economic benefits to the pipe industry when used in some sizes and strength classes. The theoretical and experimental results of this investigation suggest the following general conclusions relative to the use of this material in concrete pipe.

1. Welded deformed wire fabric provides better bond with the surrounding concrete and better control of cracking than any of the currently used non-deformed types of reinforcing.

2. Whenever 0.01-in. crack strength governs the area of non-deformed types of reinforcing required in a pipe, welded deformed wire fabric reinforcing will be more efficient for crack control, and therefore, area reductions will be warranted.

3. When ultimate flexural strength governs the required steel area of non-deformed types of reinforcing, an area reduction of about 6 percent is indicated with welded deformed wire fabric compared to conventional welded wire fabric reinforcement. This reduction is possible because the minimum specified (ASTM) ultimate tensile strength of welded deformed wire fabric is 7 percent greater than that of welded smooth wire fabric.

4. Ultimate diagonal tension strength of pipe with welded deformed wire fabric may be somewhat lower than comparable pipe with the same amount of conventional welded smooth wire fabric, because of the usual larger spacing of longitudinal wires employed with deformed material. Available test information indicates a decrease in diagonal tension strength of pipe with welded deformed wire fabric and wide spacing of longitudinals compared to the same amount of conventional fabric with 8-in. spacing of longitudinals of about 200 in ultimate D-Load.

5. For larger size pipe with strength requirements greater than Class III levels, the use of stirrup reinforcing for diagonal tension is the only practical means to achieve the specified ultimate strength. For such pipe, the quantity of circumferential steel is usually governed by the specified minimum 0.01-in. crack strength, while stirrup reinforcement inhibits premature failure in diagonal tension. In this case, welded deformed wire fabric is particularly efficient because it has a combination of high ultimate strength and outstanding crack control efficiency.

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