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Foreword

This RECORD contains three papers on the structural design and the fabrication of pipe for use in culvert and sewer construction and similar applications. The first two papers apply to rigid-type reinforced concrete pipe and the third paper deals with flexible-type aluminum alloy culvert.

In the first report, Heger and Gillespie describe the structural behavior of circular concrete pipe reinforced with welded deformed wire fabric as related to the 0.01-inch crack strength and ultimate strength requirements of ASTM Specification C 76. Three-edge bearing tests were made on 70 full-size specimens with diameters of 48 to 114 inches. Based on the test results, the authors indicate that welded deformed wire fabric controls crack width more effectively than the various other nondeformed reinforcements commonly used for concrete pipe. Where steel area requirements are governed by 0.01-inch crack strength criteria, C 76 requirements may be met with much less steel using the welded deformed wire fabric reinforcement. Because of the higher tensile strength specified for deformed wire, ultimate flexural strength is slightly higher with deformed wire than with smooth wire.

In certain sizes, stirrup reinforcing is needed to obtain the required diagonal tension strength. The authors state that the required areas of steel, being governed by the 0.01-inch crack, permit welded deformed wire fabric with a wide spacing of longitudinals making it an economical means of reinforcement. Design equations are developed to enable reasonable predictions of strength requirements for the various sizes and strength classes of ASTM C 76.

Spangler reviews the use of the three-edge bearing test requirements, both for the 0.01-inch crack and the ultimate load on which are based the concrete pipe strength specifications of the American Society for Testing and Materials and other agencies. He points out that it is not logical to define the ultimate load on this kind of pipe installed in the ground because of the development of passive-resistance pressures by the sidefill soil as the pipe deforms under heavy loading. Since no lateral pressures are applied to the pipe in the laboratory three-edge bearing test, it is impossible to translate the very severe ultimate test strength into an ultimate strength in the ground, even if it could be defined.

The author concludes that the ultimate test load requirement is no longer a useful design tool and should be discontinued as extremely expensive since it is a test of pipe to its destruction. A design is shown for pipe based on the 0.01-inch crack test strength. Suggestions are also given for extending the useful life of pipe damaged by loading and on factors of safety to be considered in design of pipe.

Koepf reviews several methods of design for flexible type culverts with specific application to the recently developed aluminum alloy pipe. He discusses practicality of using and limitations of each method. From this study, structural data including fill height tables for aluminum culverts are developed for the standard $^1/_2$ by $^2/_3$-inch and the 1 by 3-inch corrugation shapes. The author concludes that aluminum alloy culvert is structurally adequate in a wide range of sizes and placement conditions. His analysis indicates that permissible heights of cover are generally governed by seam strength in the smaller sizes and by flexibility in the larger sizes.

These papers emphasize the importance of sound practices for bedding and backfilling pipe to ~~assume~~ attainment of optimum structural properties.

ASSURE

—Kenneth S. Eff

Contents

Design of Circular Concrete Pipe Reinforced With Welded Deformed Wire Fabric

FRANK J. HEGER, Simpson Gumpertz and Heger Inc., Cambridge, Mass., and JAMES W. GILLESPIE, Manager, Marketing Technical Services, U.S. Steel Corporation, Pittsburgh, Pennsylvania

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.

Paper sponsored by Committee on Culverts and Culvert Pipe.

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 5½ in. or larger, in.

a' = Depth of equivalent rectangular stress block at ultimate strength of concrete section for pipe with wall thickness less than 5½ 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

$C_{1,2,...}$ = 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 concrere area, A_s/bd
p_e = Effective reinforcement ratio, $\pi D^2_m/4A_{cs}$
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.
ϕ_f, $\phi_{.01}$, ϕ_d, ϕ_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 SPECIME

Pipe Mark	ASTM Class-Wall	Type Reinforcement	Nominal I.D. in.	Wall Thickness: Crown in.	Invert in.	Left Springing in.	Right Springing in.	Inner Cage Reinforcement: Furnished A_{s_1} sq. in./ft.	Wire Spacing in.	Wire Size # or in.	Effective d_1 (in.) Crown	Invert	Steel Ult. Str. by Test psi
US 48- 1a	II B	WWF Def.	48	5.00	5.00	5.00	5.00	0.140	2 x 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 x 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 x 16	D3.2/8	4.21	4.18	96,185
US 48- 2b	III B	WWF Def.	48	5.06	5.06	5.00	5.00	0.185	2 x 16	D3.2/8	3.84	3.96	96,185
US 48- 3a	IV B	WWF Def.	48	4.94	5.06	5.06	4.94	0.361	2 x 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 x 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 x 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 x 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 x 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 x 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 x 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 x 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 x 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 x 8	1 x 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 x 8	1 x 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 x 8	1 x 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 x 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 x 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 x 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 x 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 x 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 x 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 x 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 x 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 x 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 x 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 x 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 x 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 x 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 x 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 x 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 x 16	D5/7	7.06	6.81	87,250
US 72-13a	IV C+	WWF Def.	72	8.00	7.94	7.94	8.00	0.484	2 x 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 x 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 x 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 x 16	D6/6	7.99	8.05	95,413
US 96- 2a	III B	WWF Def.	96	9.00[1]	9.25	8.94	9.00	0.480	2 x 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 x 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 x 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 x 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 x 16 2 x 16	1-D14/5 & 1-D11/5EII.	7.42	7.52	—
US 96- 5a	V+ B	WWF Def. + Stirrups	96	9.19	8.97	9.13	9.13	1.466	2 x 16 2 x 16	1-D14/5 & 1-D11/5EII.	7.45	6.86	83,700
US114- 1a	III B	WWF Def.	114	10.50	10.56	10.50	11.00	0.668	2 x 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 x 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 x 16	D19/5	9.37	8.62	88,400
US114- 3a	IV+ B	WWF Def. + Stirrups	114	10.50	10.69	10.81	10.94	1.122	2 x 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

		Outer Cage Reinforcement				Concrete Properties					
urnished A_{s} q. in./ft.	Wire Spacing in.	Wire Size # or in.	Effective d_2 (in.) Left Springing	Effective d_2 (in.) Right Springing	Steel Ult. Str. by Test psi	Cylinder Ave. Comp. Str. psi	Core Ave. Comp. Str. psi	Combined[3] Ave. Comp. Str. psi	Condition of Reinforcing	Age of Pipe at Test days	Pipe Mark
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[2]	5.90[2]	98,760	4757	4782	4480	L.R.	22	US 72-4a
0.191	2 x 16	D3.2/8	5.96[2]	5.90[2]	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	[illegible].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 &	7.30	7.62	—	5630	6292	5520	H.R.	20	US 96-4a
	2 x 16	1-D11/5EII.									
1.320	2 x 16	1-D11/5 &	6.93	6.81	87,840	7140	7814	6940	L.R.	35	US 96-5a
	2 x 16	1-D11/5EII.									
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.

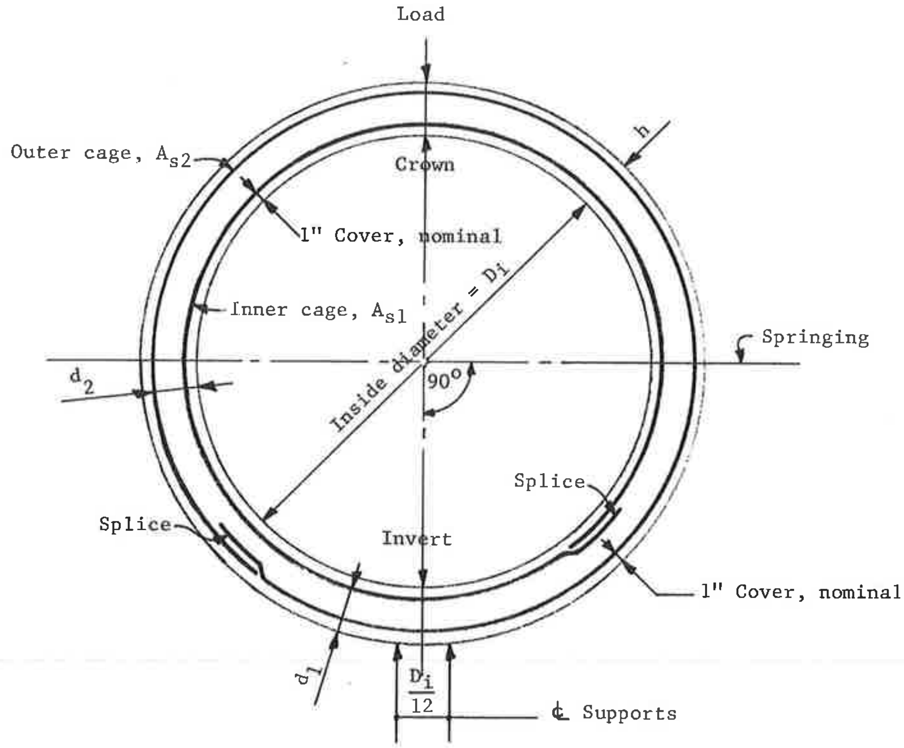

Figure 1. Pipe test specimens—typical transverse section.

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

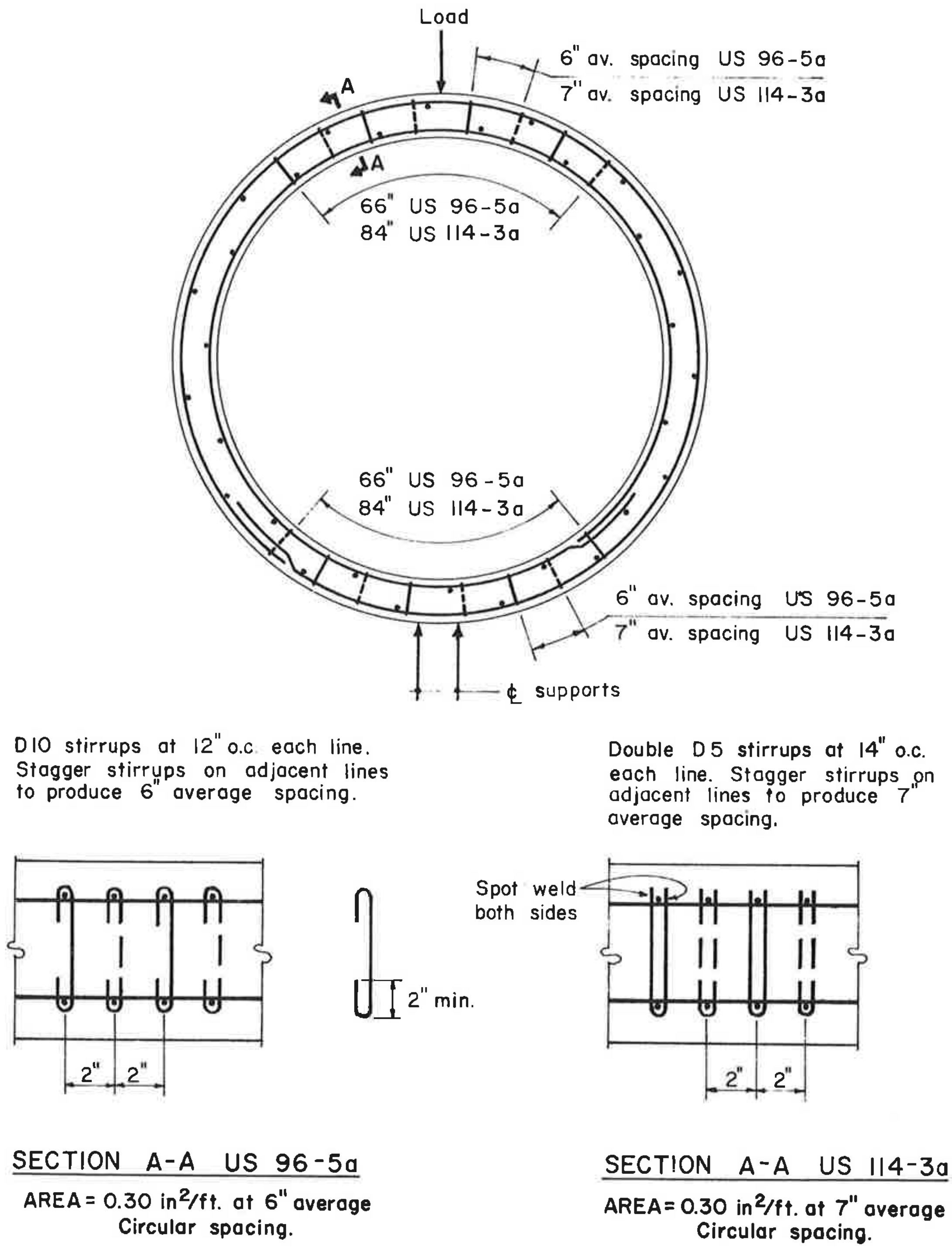

Figure 2. Stirrup details.

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DESCRIPTION OF TEST SPE

Pipe Mark	ASTM Class-Wall	Type Reinforcement	Nominal I. D. in.	Wall Thickness Crown in.	Wall Thickness Invert in.	Wall Thickness Left Springing in.	Wall Thickness Right Springing in.	Inner Cage Reinforcemen Furnished A_{s1} sq. in./ft.	Wire Spacing 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.

2

IS—OTHER TEST PROGRAMS

Steel Ult. Str. by Test psi	Outer Cage Reinforcement: Furnished A_{s2} sq. in./ft.	Wire Spacing in.	Wire Size # or in.	Effective d_2 (in.) Left Springing	Effective d_2 (in.) Right Springing	Steel Ult. Str. by Test psi	Age of Pipe at Test days	Cylinder Ave. Comp. Str. psi	Pipe Mark
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 i2	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	31	7180	MS 72-1a
92,097	0.267	2 x 16	D4.5/8	5.38	5.38	98,430	31	7480	MS 72-1b
92,097	0.267	2 x 16	D4.5/8	5.13	5.13	98,430	34	7070	MS 72-1c
92,173	0.290	2 x 16	D5/7	6.37	6.37	87,250	31	7730	MS 84-1a
92,173	0.290	2 x 16	D5/7	6.87	6.87	87,250	31	7780	MS 84-1b
92,173	0.290	2 x 16	D5/7	7.12	7.12	87,250	34	7305	MS 84-1c
91,370	0.354	2 x 16	D6/6	7.36	7.36	92,097	31	6985	MS 96-1a
91,370	0.354	2 x 16	D6/6	7.86	7.86	92,097	31	8135	MS 96-1b
91,370	0.354	2 x 16	D6/6	8.11	8.11	92,097	34	7250	MS 96-1c
83,000	0.432	2 x 12	D7.2/5	7.98	7.98	93,500	13	5185	MS 96-2a
83,000	0.432	2 x 12	D7.2/5	7.85	7.85	93,500	13	4213	MS 96-2b
83,000	0.432	2 x 12	D7.2/5	7.85	7.85	93,500	13	4127	MS 96-2c
90,837	0.484	2 x 16	D8/5	7.59	7.59	91,370	31	7410	MS 114-1a
90,837	0.484	2 x 16	D8/5	7.34	7.34	91,370	31	7180	MS 114-1b
90,837	0.484	2 x 16	D8/5	7.34	7.34	91,370	34	7835	MS 114-1c
97,000	0.590	2 x 12	D9.9/5	8.32	8.32	86,000	13	4880	MS 114-2a
97,000	0.590	2 x 12	D9.9/5	8.32	8.32	86,000	13	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.

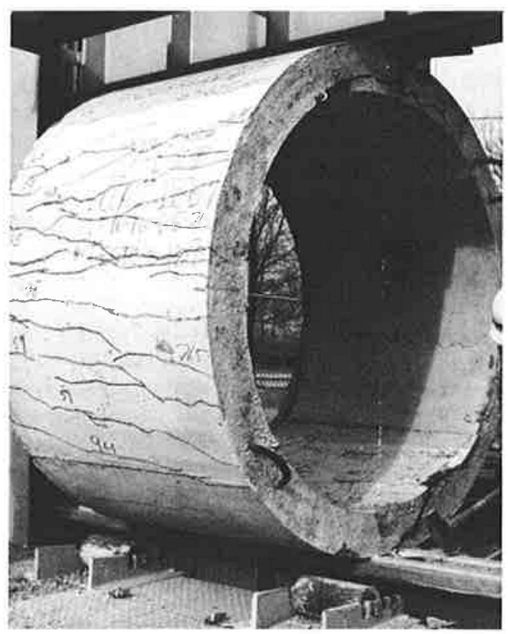

Figure 4. Crack pattern and mode of failure for 72-in. pipe.

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 occurence 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[4]	D-Load 0.01" Crack Test	$\frac{DL_{.01\ test}}{DL_{.01\ design}}$	D-Load Ultimate Design[5]	D-Load Ultimate Test	$\frac{DL_{u\ test}}{DL_{u\ design}}$	Average Crack Spacing at 0.01" Crack Load (in.) Crown	Average Crack Spacing at 0.01" Crack Load (in.) Invert	Mode of Failure
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 49- 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[2]	875	2000	1583	0.79[2]	3000	2000	0.67[2]	6	4	D. T.
US 72- 9b[2]	917	2000	1208	0.60[2]	3000	2313	0.77[2]	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[3]	1684	3000	3052	1.02	3750	4820+[1,3]	1.29+[1,3]	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[3]	1158	2200	2632	1.19	3300	4552[3]	1.37[3]	5-1/4	5-3/4	R. T.

Notes:

1. Capacity of testing machine reached at $(DL)_u = 4816$.
2. This was a special test for low concrete strength.
3. Stirrup reinforcing used.
4. "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.
5. 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[1]	D-Load .01" Crack Test	$\frac{DL_{.01\ test}}{DL_{.01\ design}}$	D-Load Ultimate Design[2]	D-Load Ultimate Test	$\frac{DL_{u\ test}}{DL_{u\ design}}$	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[3]	1350	1620	1.20	2000	1900	0.95[3]	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[4]	1650	2080	1.26	2450	2930	1.20[4]	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.

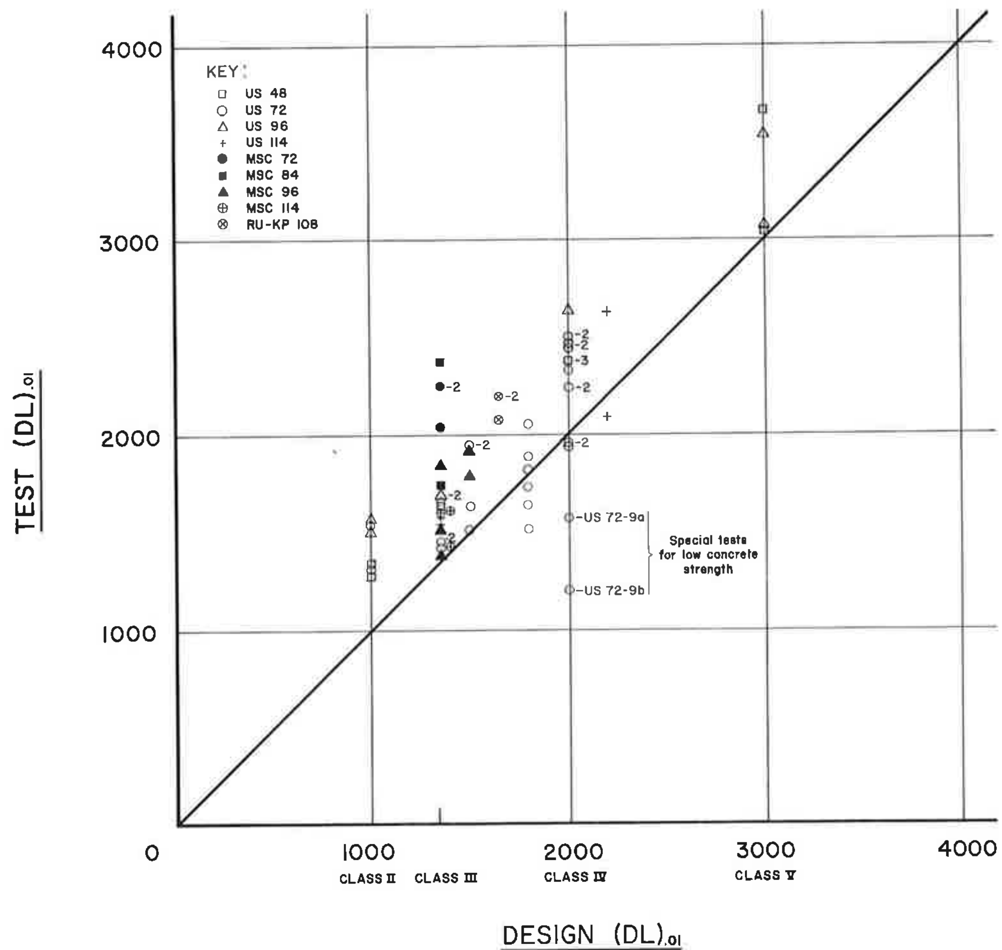

Figure 6. Comparison of test and required C 76 0.01-in. crack strengths.

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:

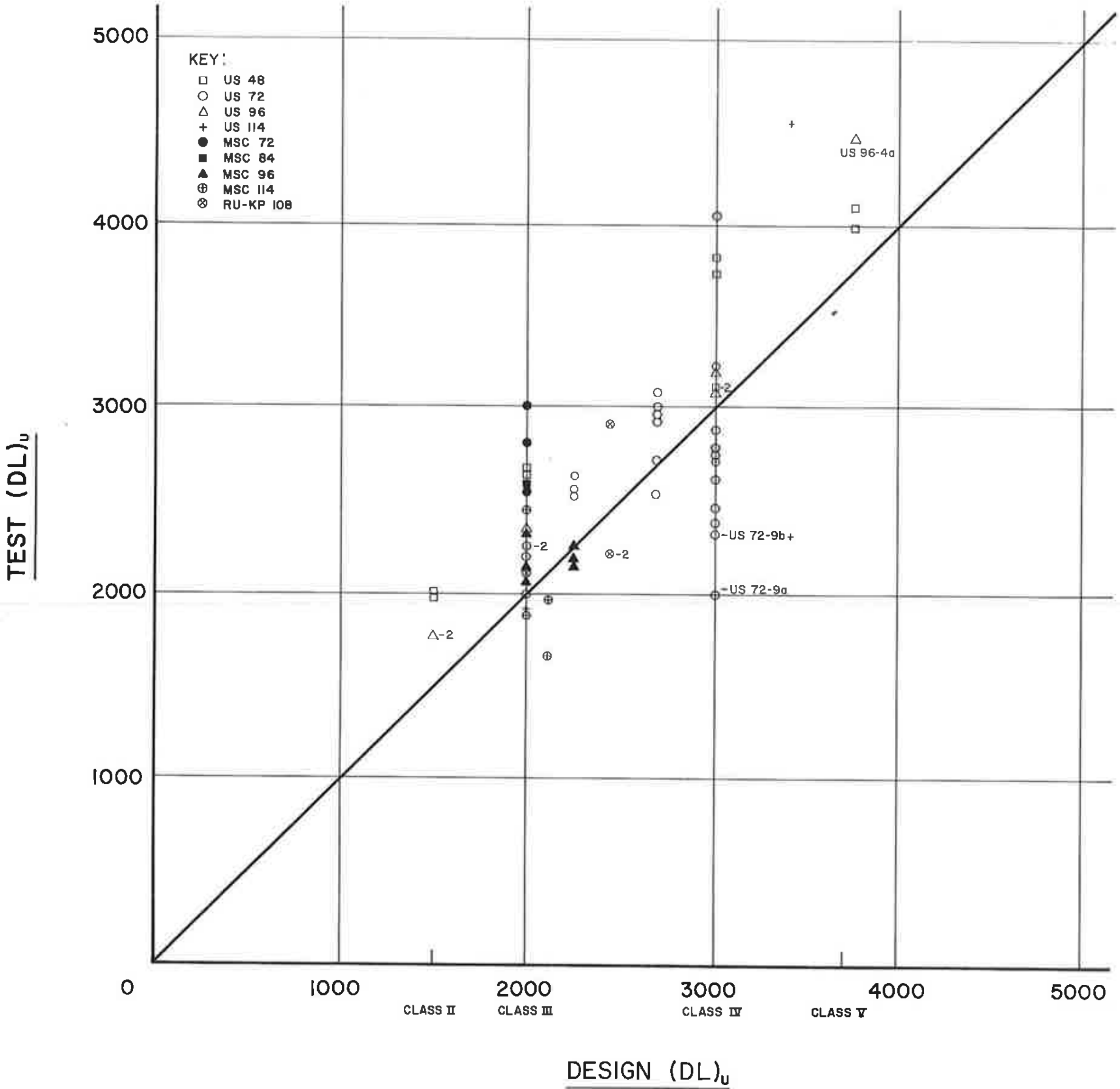

Figure 7. Comparison of test and required C 76 ultimate strengths.

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):

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

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 A 305 type deformed bars:

$$w_{max} = 11.5 \times 10^{-8} f_s \sqrt[4]{A_{cs}} \tag{2}$$

Cornell University (10), based on A 305 type deformed bars:

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

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

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

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 A 305 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:

$$DL_{.01} = \frac{6.2 \times 10^6 A_{s1} d_1}{\sqrt[4]{A_{cs}}\, D_i^2} - 9 \frac{W}{D_i} \tag{5}$$

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:

$$DL_{.01} = C_1 \sqrt{f'_c} + \frac{C_2 A_{s1} d_1}{\sqrt[4]{A_{cs}}\, D_i^2} - 9 \frac{W}{D_i} \tag{6}$$

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

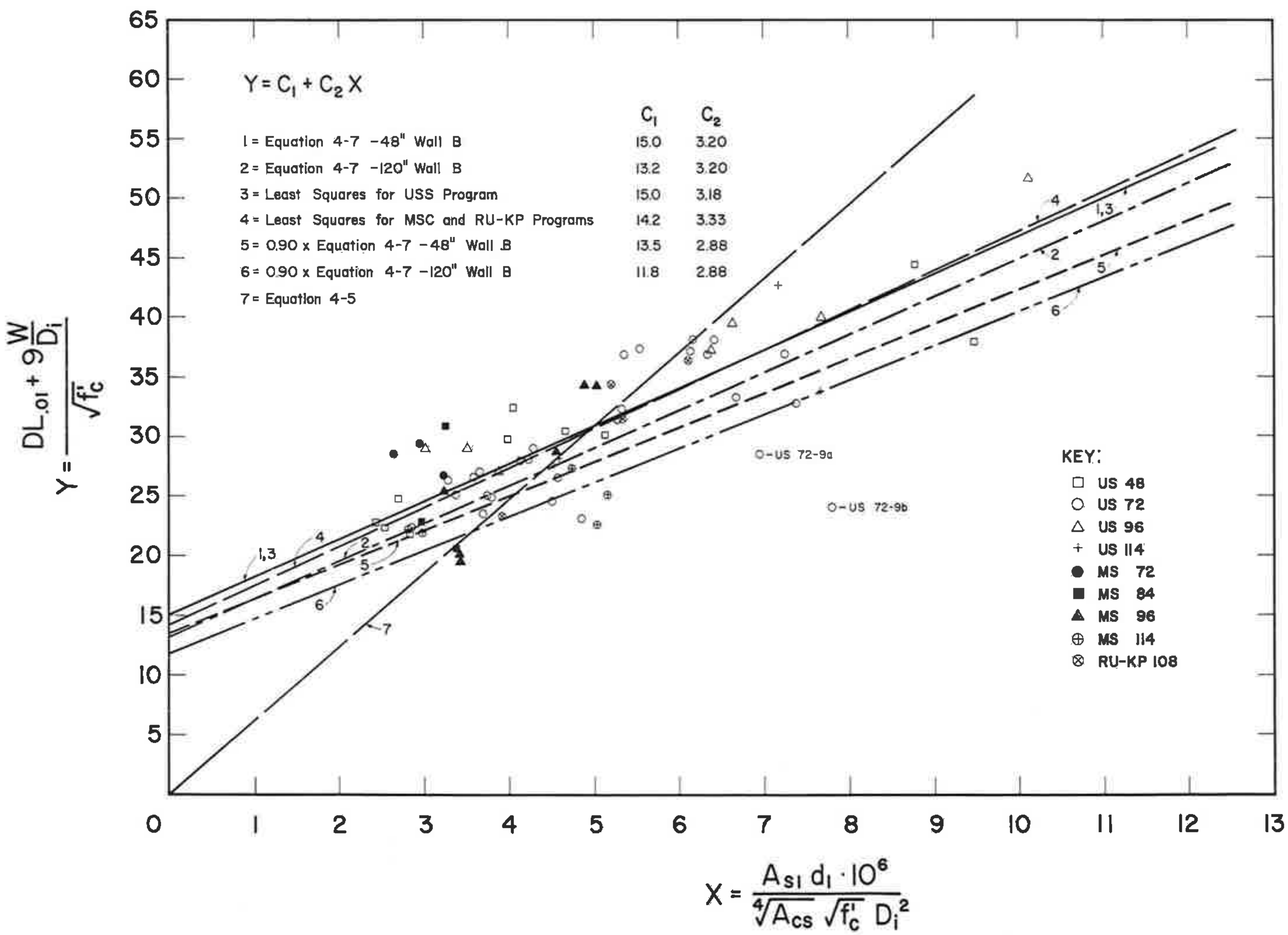

Figure 8. Development of 0.01-in. crack D-Load strength equation.

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

$$C_1 = 180$$

$$C_2 = 3.18 \times 10^6$$

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:

$$DL_{.01} = \frac{144\, h \sqrt{f_c'}}{D_i} + \frac{3.2 \times 10^6 A_{s1}\, d_1}{\sqrt[4]{A_{cs}}\, D_i^2} - 9\, \frac{W}{D_i} \qquad (7)$$

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 $f_{s.01} = f_y$):

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

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 5½ in. or larger having two lines of welded (deformed or smooth) wire fabric reinforcing:

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

where

$$a = 0.1\,\frac{f_{su1}\, A_{s1}}{f_c'} \qquad (10)$$

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

For pipe with wall thickness less than 5½ in. having two lines of welded (deformed or smooth) wire fabric reinforcing:

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

where

$$a' = 0.175\,\frac{f_{su1}\, A_{s1}}{f_c} \leq 0.8 \text{ in.} \qquad (13)$$

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.

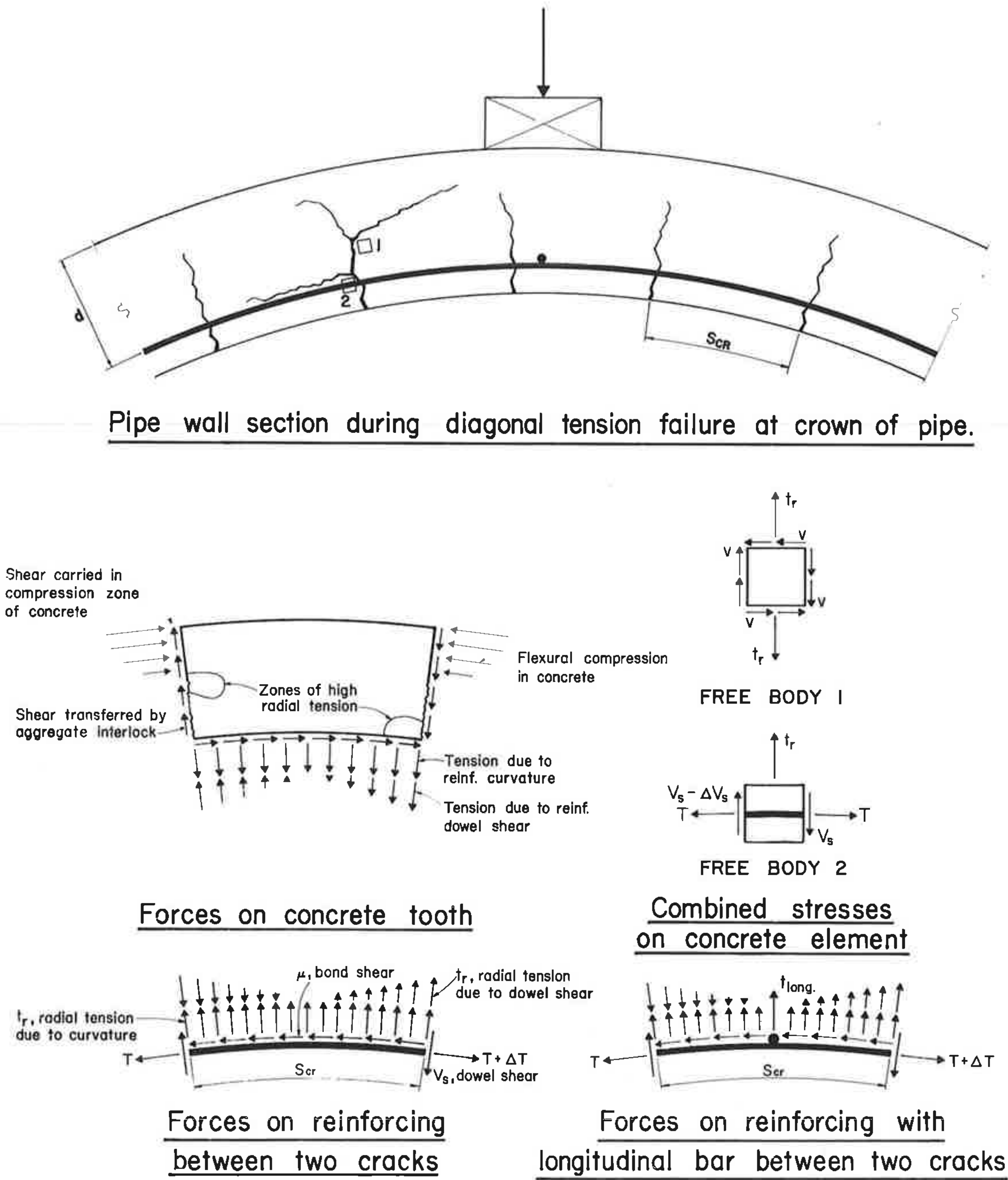

Figure 9. Mechanism of diagonal tension failure.

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:

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

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:

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

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:

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

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.

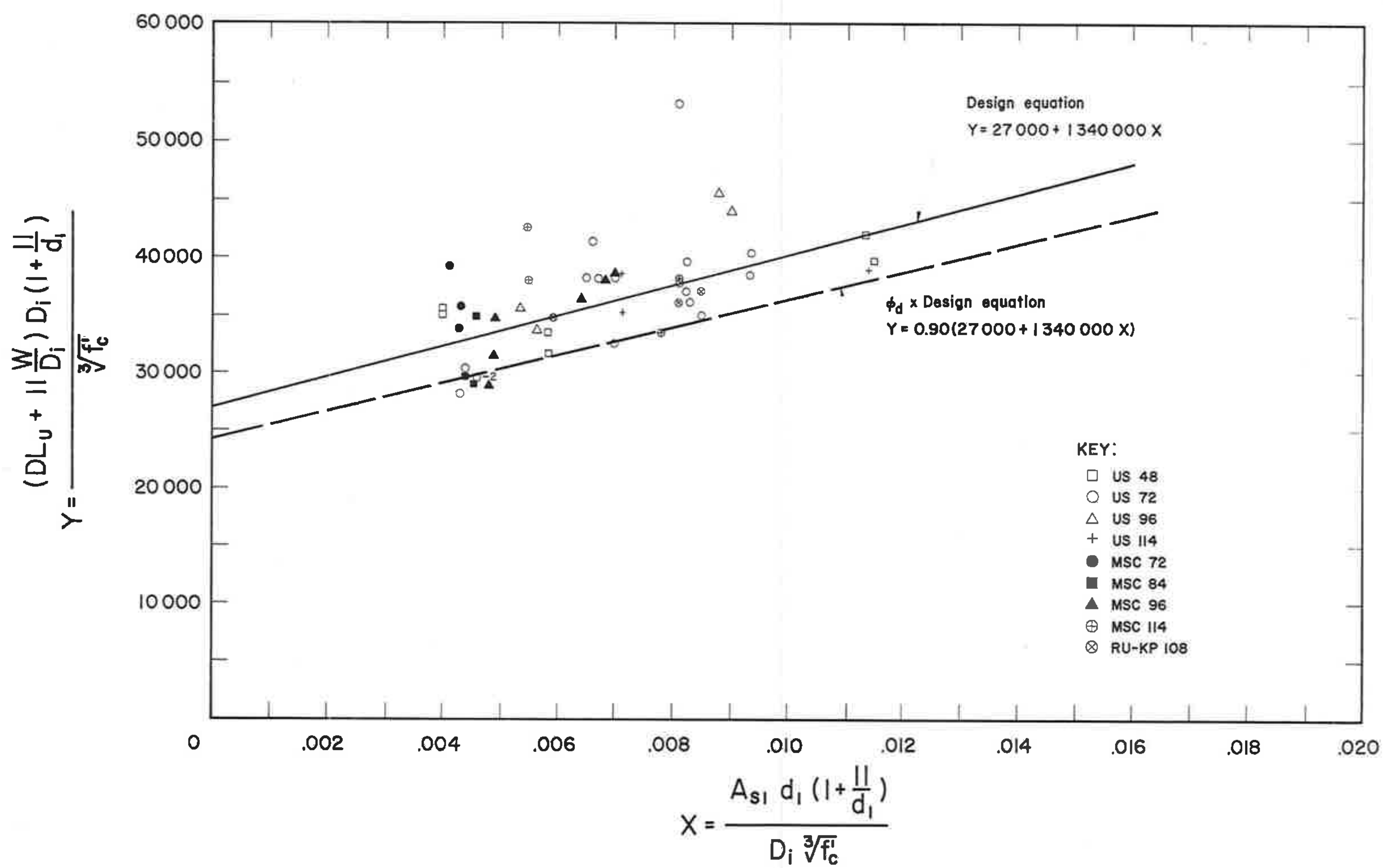

Figure 10. Development of diagonal tension D-Load strength equation.

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):

$$DL_u = \frac{450\, d_1 \sqrt{f_c'}}{D_i} + \frac{1{,}120{,}000\, A_{s1}\, d_1}{D_i^2} - 11\frac{W}{D_i} \tag{39}$$

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:

$$DL_u = \left[\frac{27{,}000\, d_1 \sqrt[3]{f'_c}}{D_i\,(d_1 + 11)} + \frac{1{,}340{,}000\, d_1\, A_{s1}}{D_i^2} + C_L N_L \right] \frac{L_e}{L_n}\, \phi_d - 11\, \frac{W}{D_i} \qquad (17)$$

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,

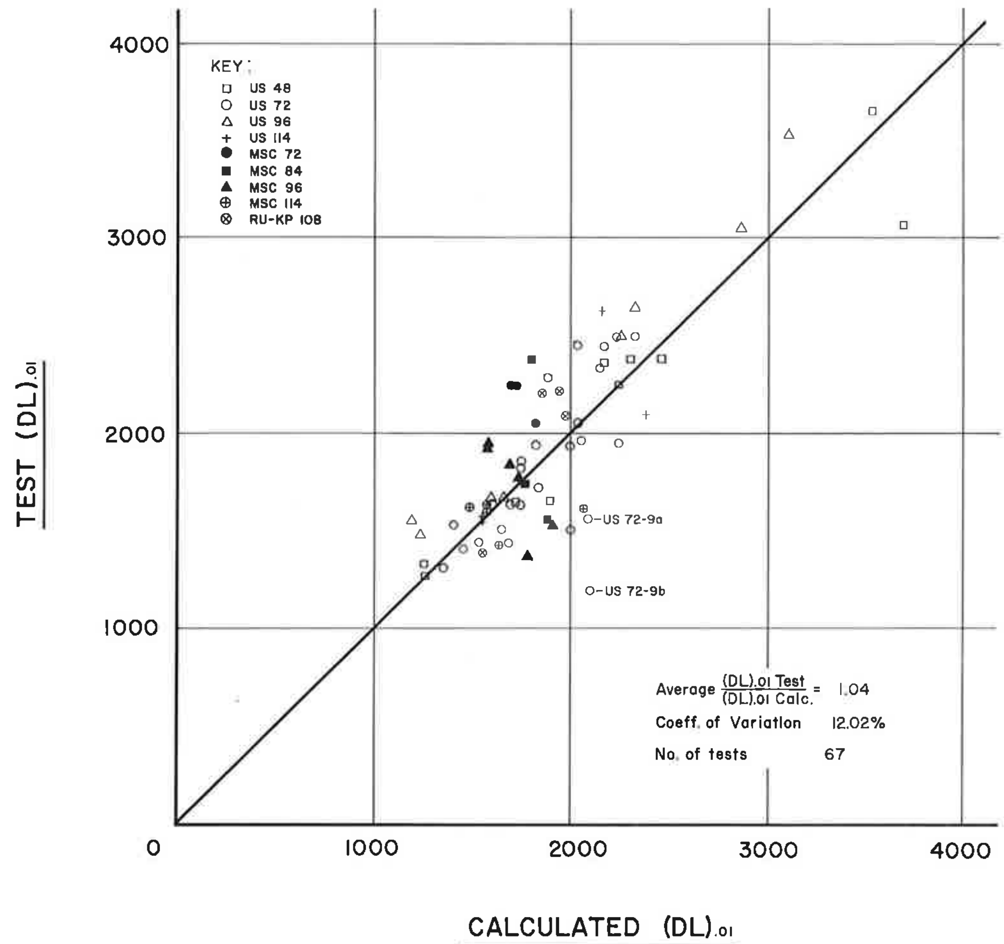

Figure 11. Comparison of test and calculated 0.01-in. crack strengths.

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 $DL_{.01}$ test/$DL_{.01}$ 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 $DL_{.01}$ test/$DL_{.01}$ 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 $DL_{.01}$ test/$DL_{.01}$ 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 $DL_{.01}$ test to $DL_{.01}$ 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 test/DL_u calc used is that which corresponds to the observed failure mode. For the nine test specimens which failed in flexure, the average DL_u test/DL_u calc = 0.98. Coefficient of variation is 6.53 percent. For the 57 test specimens which failed in diagonal tension, the average DL_u test/DL_u 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.

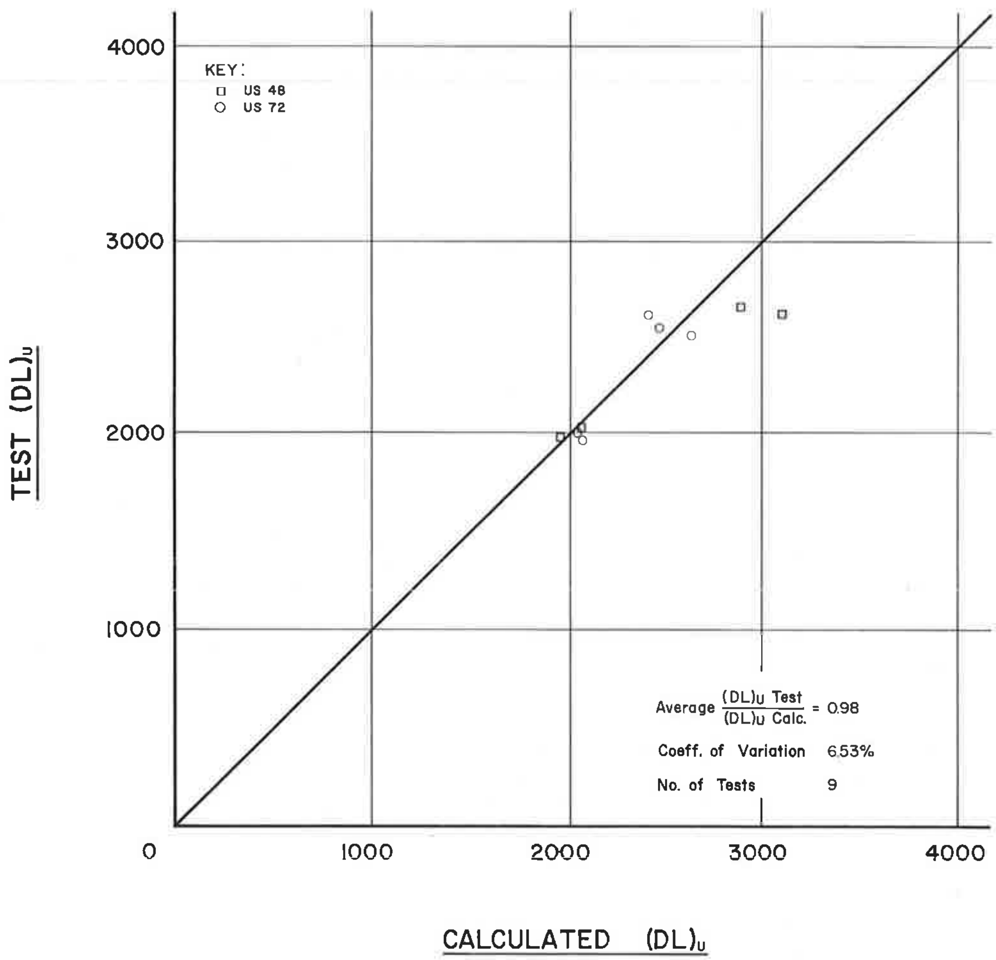

Figure 12. Comparison of test and calculated ultimate flexural strengths.

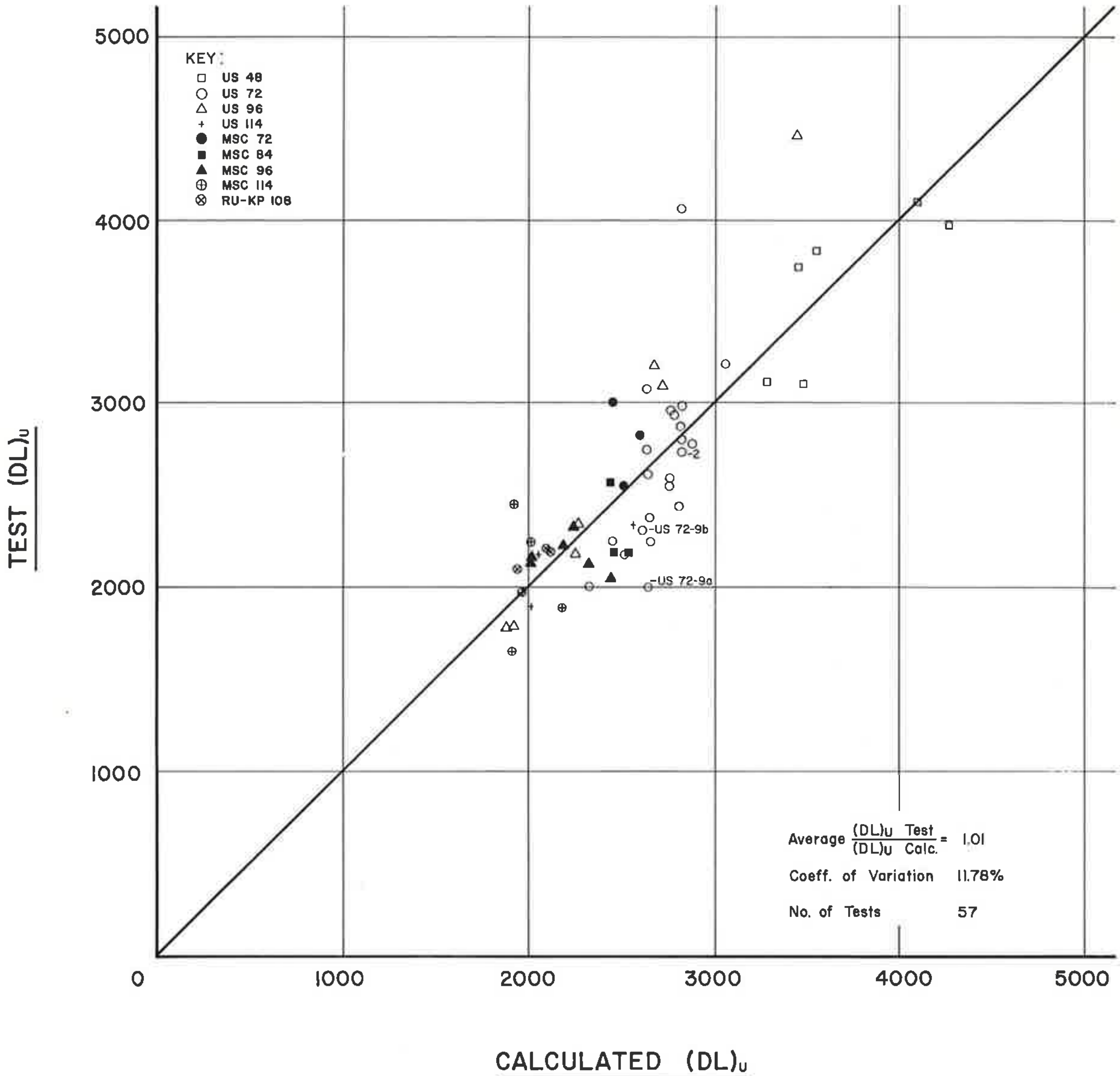

Figure 13. Comparison of test and calculated ultimate diagonal tension strengths.

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 Str. by Test[2] ksi	Ultimate Str. by Test ksi	at $(DL)_{.01}$ ksi	at $(DL)_{u}$ 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[1]	101.0	55.0	86.1*
US 72-1b	85.0[1]	101.0	57.1	81.0
US 72-1c	84.6[1]	101.0	50.2	79.4
US 72-2a	90.9[1]	95.3	56.2	87.6
US 72-2b	90.9[1]	94.5	57.9	80.2
US 72-2c	90.9[1]	94.0	45.6	83.6
US 72-3a	70.5[1]	78.4	70.5	88.7*
US 72-3b	67.6[1]	75.1	59.2	94.4*
US 72-3c	68.0[1]	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: Yield Str. by Test[2] ksi	Steel Properties: Ultimate Str. by Test ksi	Calculated Max. Steel Stress Elastic Theory: at $(DL)_{.01}$ ksi	Calculated Max. Steel Stress Elastic Theory: at $(DL)_U$ ksi
RU-KP 108-1a	85.2[1]	94.7	58.7	58.7
RU-KP 108-1b	85.2[1]	94.7	59.8	59.8
RU-KP 108-2	85.2[1]	94.7	56.2	76.1
RU-KP 108-3	81.0[1]	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 ϕ_f, ϕ_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, ϕ_d and $\phi_{.01} = 0.90$ are suggested for design to meet these strength requirements.

4. Determine a pipe fabrication and materials variability factor, ϕ_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 ϕ_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, ϕ_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 5½ 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:

$$a = 0.1 \frac{f_{su}}{f_c'} A_{s1} \tag{18}$$

(2) Determine required steel area:

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

(b) For pipe with wall thickness less than 5½ in. with outer cage area three-fourths of inner cage area,

(1) estimate trial values for f_c' and A_{s1} and calculate:

$$a' = 0.175 \frac{f_{su}}{f_c'} A_{s1} \tag{20}$$

(2) Determine required steel area:

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

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:

$$A_{s1} = \frac{3.1 \times 10^{-7}\, D_i^2 \sqrt[4]{A_{cs}} \left[\left(DL_{.01} + 9\, \frac{W}{D_i}\right) \frac{1}{\phi_{.01}} - \frac{144\, h\, \sqrt{f_c'}}{D_i}\right]}{d\, \phi_x} \tag{22}$$

Check:

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

(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:

$$DL_u = \left[\frac{27{,}000\, d \sqrt[3]{f'_c}}{D_i\,(d + 11)} + \frac{1.34 \times 10^6\, d\, A_{s1}\, \phi_x}{D_i^2} + C_L N_L\right] \frac{L_e}{L_n}\, \phi_d - 11\, \frac{W}{D_i} \tag{24}$$

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:

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

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

11. Determine outer cage steel area:

$$A_{s2} \geqq 0.75\, A_{s1} \tag{26}$$

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. ± $^3/_8$ 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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The Case Against the Ultimate Load Test for Reinforced Concrete Pipe

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•THE American Society for Testing and Materials and other specification writing agencies have, for many years, published standard specifications for the design and fabrication of reinforced concrete pipe for use in culvert and sewer construction and allied fields. From the very first ASTM tentative specification issued in 1930 to the current standard C 76-65 T, these specifications have included requirements relative to wall thickness, amount and disposition of reinforcing steel, quality of materials, manufacturing tolerances and the like. In addition, the structural quality of the manufactured product was specified in terms of required strengths which representative specimens of the pipe must meet when loaded in a laboratory crushing strength test. Up until the current standard, either of two types of prescribed strength tests were permitted—the Sand Bearing Test or the 3-Edge Bearing Test. By action of ASTM Committee C-13 in 1964, the Sand Bearing Test was eliminated from the specification.

Prior to issuance of the earliest strength specification, it was generally required that a reinforced concrete pipe meet both of two separate and distinct minimum test load criteria—the first crack strength and the ultimate strength. First crack strength was defined as the test load at which the first visible crack appeared in the pipe wall, usually a longitudinal crack on the inside of the pipe at the invert, though frequently at the crown and invert simultaneously. Ultimate strength was defined as the maximum or ultimate test load which the pipe could sustain.

The late Professor W. J. Schlick of Iowa State University had extensive experience in conducting tests of reinforced concrete pipe. He observed that some difficulty arose in determining the test load at "first crack." Light conditions in the laboratory, color and surface texture of the test specimen, and even the visual acuity of the observer, all entered into the decision as to when the first crack occurred. In order to provide for a more definite criterion for determining the test strength at an early stage of visible load effect, he suggested that a crack 0.01 in. wide be substituted for the first crack. This provided for a positive criterion which could actually be measured by means of a mechanic's leaf gage, thus eliminating most of the uncertainties associated with the first crack load requirement. His suggested modification was incorporated in the first ASTM tentative standard and has remained in the specification up to the present.

As stated above, the early specifications provided that the pipe must comply with both the 0.01-in. crack strength and the ultimate strength requirements. This provision was modified in 1957 to allow acceptance of the pipe on the basis of the 0.01-in. crack strength alone, or on the basis of both the 0.01-in. crack strength and the ultimate strength, at the option of the purchaser. This modification was made very largely in the light of experience accumulated in the Pacific Coast region, where the design of pipes and pipe installations is primarily based only on the 0.01-in. crack strength. Both strength requirements are still widely used in other regions of the country.

This history of the development of the ASTM standard specifications for reinforced concrete pipe is presented to show that, with the exception noted in the preceding paragraph, the ultimate load test has been a part of the strength requirements for this

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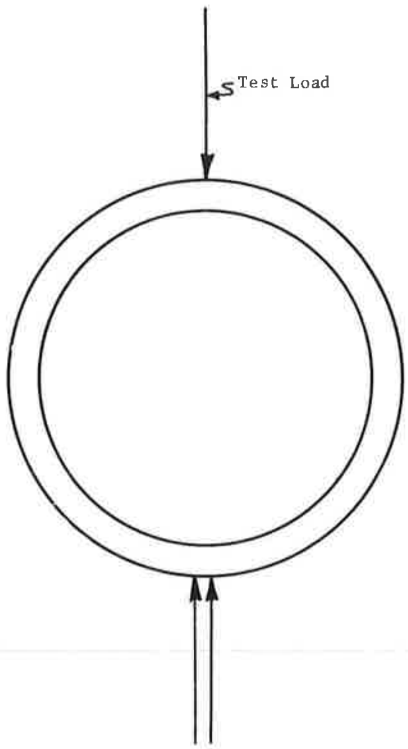

Figure 1. Load system, 3-edge bearing test.

product from the very beginning. The author believes that the test for ultimate strength has outlived its usefulness and recommends that it be eliminated from specifications. The purpose of this paper is to outline the procedure for designing a reinforced concrete pipe installation, and to point out the fact that the ultimate test strength does not serve a useful purpose in connection with that procedure. Nor is it indicative of the load-carrying capacity of a pipe when installed in the ground. Since it is necessarily a test to destruction, it is an expensive test and should be discontinued. This is especially true in the case of large-diameter pipes, which are being used more and more widely, both in highway and sewer construction.

The 3-edge bearing test is a very severe load test. The load system on the pipe specimen consists of the applied vertical load concentrated along a longitudinal element at the top, and an equal and opposite reaction concentrated along two closely spaced longitudinal elements at the bottom (Fig. 1). There are no lateral pressures applied to the pipe during the test. Bending moments in the pipe wall are relatively high because of the concentrated load; reaction and test strength values, both the 0.01-in. crack and the ultimate, are correspondingly low.

In contrast, when a pipe is installed in the ground, the system of loads acting on it is usually much more favorable. As a generalization, the earth load on top is distributed approximately uniformly over the horizontal width—the outside diameter—of the structure. The bottom reaction is distributed laterally over some fraction of the horizontal diameter, depending on the kind and quality of the bedding in which the pipe is installed. In addition, under favorable circumstances, active lateral earth pressures may act against the sides of the pipe. Lateral pressures tend to produce bending moments in the pipe wall which are in the opposite direction from those induced by vertical loads. Therefore every pound of lateral pressure which reliably can be brought to bear against the sides of a pipe increases its capacity to carry vertical load approximately one for one.

The strength design of a specific pipe installation follows the same classical pattern as that of any other type of structure. First it is necessary to determine the maximum load to which the pipe will be subjected during its functional life. Then the designer selects the materials and the type of installation environment which will insure that the pipe will adequately support this maximum load, with a reasonable factor of safety.

The load-carrying capacity of a reinforced concrete pipe in a field installation may be determined by multiplying its 3-edge laboratory test strength by an appropriate load factor which is defined as the ratio of the pipe supporting strength under any stated condition of loading to its 3-edge bearing strength. Load factors for various installations depend on the distribution of the load on top of the pipe, the distribution of the bottom reaction, and the magnitude and distribution of active lateral pressures on the sides of the pipe. Several field loading systems and corresponding load factors are illustrated in Figure 2.

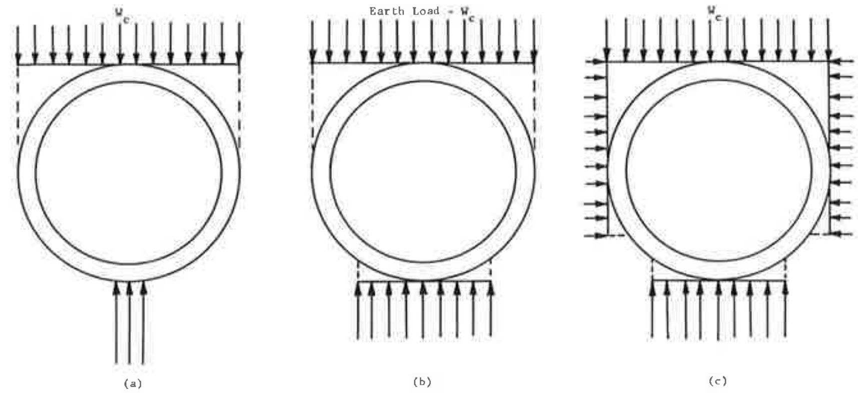

Figure 2. Load systems, various pipe installations: (a) impermissible bedding (Class D), load factor = 1.1; (b) ordinary bedding (Class C), load factor = 1.5; (c) ordinary bedding with active lateral pressure, load factor usually greater than 2.0 (See Eqs. 25-1 and 25-2, p. 424, Ref. 3).

When a reinforced concrete pipe is loaded by earth overburden in the field, the pipe deforms, i.e., the vertical diameter shortens and the horizontal diameter lengthens. The amount of this deformation in early stages of loading is very small because of the inherent rigidity of the pipe. As the load increases, the stress in the reinforcing steel increases, and since the modulus of elasticity of steel is much greater than that of concrete, the protective cover of concrete begins to show fine longitudinal cracks in the tensile zones on the inside of the pipe before the steel is stressed up to its capacity. Such cracks, in the opinion of the author, are not to be considered detrimental to the integrity of the pipe unless or until they approach a width which will permit or promote corrosion of the reinforcing steel. At the present time it is rather widespread practice to consider 0.01 in. as the limiting width of crack which can be tolerated, but there is need for extensive research to determine widths of cracks in the concrete which will effectively inhibit corrosion of the steel reinforcement under various environments.

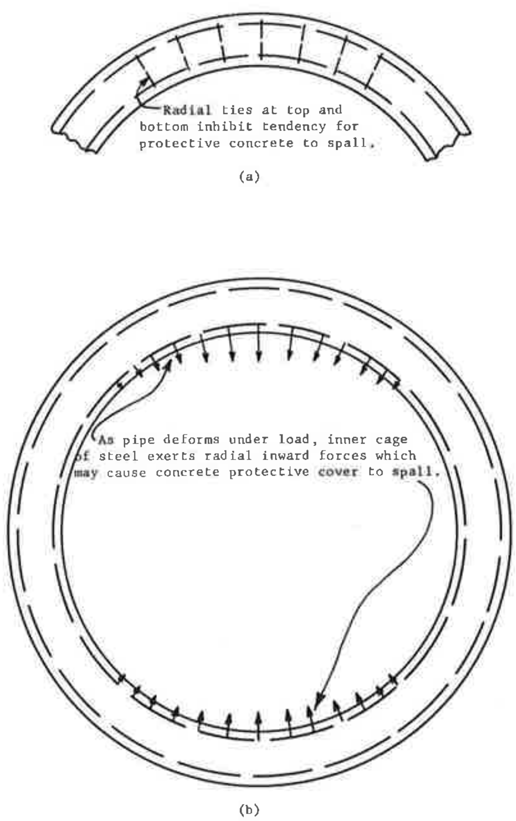

Figure 3.

As load on a pipe increases, further evidence of its effect may take the form of a separation of the protective cover of concrete from the body of the pipe wall. This separation occurs at the circumferential surface, which contains the inner layer of reinforcement, and in the tensile regions, which are at the top and bottom of the pipe. It is caused primarily by the fact that the inner cage of steel, being more flexible than the concrete wall in which it is embedded, tends to change shape more rapidly than the more rigid

Figure 4. Longitudinal crack about 1/16 to 1/8 in. wide; easily repaired by chipping and guniting after sufficient passive soil pressures have developed to establish equilibrium.

Figure 5. Concrete protective cover beginning to spall; can usually be repaired by chipping and guniting after sufficient passive soil pressures have developed to establish a state of equilibrium. Extreme cases may require pressure grouting to improve bedding conditions and lateral pressures.

concrete. The inner cage pulls downward at the top and upward at the bottom. This introduces tensile stresses in the concrete which are directed radially inward (Fig. 3b), and the protective cover may break loose. Further increase in load causes this concrete to shatter and "slab off," and the steel is laid bare, as shown in Figures 5 and 6. This type of action can be inhibited and the strength of the pipe increased by installing radial ties between the inner and outer cages of steel at the crown and invert of the pipe (Fig. 3a). The primary function of these radial ties (sometimes referred to as bridging, stirrups, or shear steel) is to hold the inner cage of reinforcement in place and prevent the development of radial tensile stress in the protective cover of concrete.

While the action described is progressing with further increase in load, the pipe loses rigidity and approaches the condition of a flexible or semirigid structure. The horizontal diameter increases under load to such an extent that the passive resistance pressure of the soil is mobilized in much the same manner as in the case of a flexible metal pipe. The primary source of supporting strength of the originally rigid structure gradually shifts from its inherent strength characteristics to dependence upon the passive resistance of the enveloping soil.

The more a pipe deforms the greater the magnitude of the mobilized passive pressure for a given soil, and it is impossible to define an ultimate pipe strength under field loading conditions in the same sense or which is comparable to the ultimate laboratory test strength, wherein no lateral pressures are applied during the test. A pipe under earth loading may have undergone gross deformation, but because of the passive soil pressures developed may still be capable of accepting additional load, and an

Figure 6. Advanced stage of spalling; note how reinforcement has pulled away from concrete wall. This 108-in. pipeline was repaired by pressure grouting through holes drilled at lower quarter points, after which new concrete protective cover was applied. Line has served 18 years since repairs were made and gives promise of a long satisfactory life.

"ultimate" load is practically never reached. Therefore it is impossible to apply a load factor to the ultimate test strength to obtain a field supporting strength, because the two strengths being considered are not comparable. They are radically different and attributable to different sources. Of course, long before the condition of gross deformation referred to is reached, the concrete protective cover will be badly shattered and the reinforcement pulled loose in the crown and invert (Figs. 5 and 6), which further complicates any attempt to define ultimate strength under field load conditions.

The author believes and recommends that the most appropriate and, in fact, the only rational approach to the design of a reinforced concrete pipe installation is to utilize the 0.01-in. crack test strength (or some similar visible and measurable indicator of early load effect), as a basis of selection of pipe strength and specified bedding and backfilling requirements. The ultimate 3-edge bearing test strength has no meaning in terms of field performance, and there is no basis for translating the results of the laboratory test into an ultimate strength under field conditions. Furthermore, since the test to ultimate requires destruction of the test specimen, and is therefore expensive to conduct, it is recommended that the ultimate strength requirement be deleted from specifications for reinforced concrete pipe. In contrast, the 0.01-in. crack strength test (or a similar width criterion) is nondestructive in character, and therefore many more pipe sections could be tested to this criterion for the same expenditure of laboratory funds. This would make laboratory testing more palatable to manufacturers and consumers alike. Many more specimens could be tested and, in the opinion of the author, this would tend to upgrade the whole process of design, manufacture and installation of reinforced concrete pipe structures.

DESIGN CALCULATION

An example of the design of a reinforced concrete pipe culvert installation based on the 0.01-in. crack 3-edge bearing strength of the pipe is presented (see ch. 24 and 25 of Ref. 3).

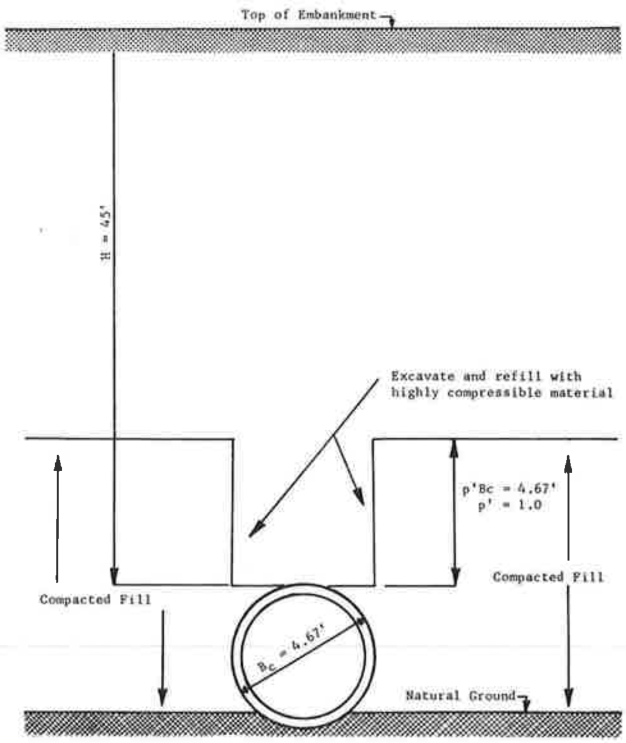

Figure 7. Example culvert, imperfect ditch installation.

Load calculation

Assume $w = 120$ pcf, $K\mu = 0.13$

$p' = 1.0$, $r_{sd} = -0.3$

$B_c = 4.67$, $\frac{H}{B_c} = \frac{45}{4.67} = 9.6$

$C_n = 5.9$

$W_c = 5.9 \times 120 \times (4.67)^2 = 15,400$ plf

Strength calculation

Assume $m = 0.7$, $x = 0.594$, $K = 0.33$

Class C bedding, $N = 0.840$

$$q = \frac{0.7 \times 0.33}{5.9}(9.64 + 0.35) = 0.390$$

$$L_f = \frac{1.431}{0.840 - (0.594 \times 0.390)} = 2.35$$

Required 0.01-in. 3-edge strength $= \frac{15,400}{2.35} = 6600$ plf

Required D-Load $= \frac{6600}{4} = 1650$ D

Use Class IV pipe (2000 D at 0.01-in. crack)

Factor of safety based on 0.01-in. crack $= \frac{2000}{1650} = 1.2$

The factor of safety based on the 0.01-in. crack strength of the pipe in the above example is 1.2. Since an appropriate factor of safety cannot be determined rationally by principles of mechanics, it remains purely a matter of judgment based on experience and observation. It is the author's opinion that a minimum factor of safety of 1.0 is both adequate and economical for reinforced concrete pipelines designed on the basis of the 0.01-in. crack strength. Reasons for this opinion are (a) the failure of this type of structure does not involve the safety of human life; (b) reinforced concrete pipes have a large reservoir of load-carrying capacity beyond the 0.01-in. crack stage, due to inherent strength and the strength imparted by passive soil pressures as the pipe deforms; and (c) a pipe in the ground does not fail suddenly or collapse completely, so there is adequate time and opportunity for making repairs in case of accidental overloading.

The ASCE Manual of Practice No. 37, "Design and Construction of Sanitary and Storm Sewers" (WPCF Manual No. 9), recommends the use of a factor of safety of 1.5 based on the ultimate test strength of the pipe. It is pointed out that this value gives exactly the same result as the value of 1.0 based on the 0.01-in. crack strength in the case of ASTM Classes I, II, III, and IV pipe, since the required ultimate strength for these classes is 1.5 times the crack strength. For Class V pipe the required test strengths are 3750 D and 3000 D respectively. Therefore, a factor of safety of 1.5 based on ultimate is the equivalent of 1.2 based on 0.01-in. crack strength. However, since the ultimate test strength of a reinforced concrete pipe has no equivalent or comparable counterpart when the pipe is installed in the ground, factors of safety based on ultimate test strength have no numerical meaning.

REPAIR METHODS

A matter of collateral interest in connection with reinforced concrete pipes which have developed structural difficulty (Figs. 4, 5, and 6) has to do with methods of repair. Some engineers and contractors have resorted to threading a metal pipe liner

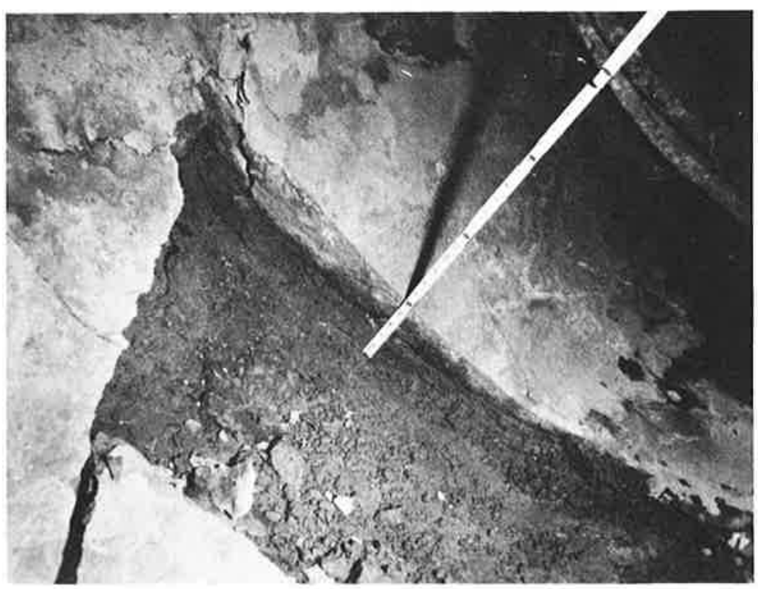

Figure 8. An 84-in. pipeline repaired by pressure grouting; several pipe sections were removed to observe grout distribution. Line has served 16 years since repairs were made and is in good condition.

of smaller diameter through the distressed pipe and grouting the annular space between the liner and the concrete. This procedure is expensive and adds the further disadvantage of reducing the hydraulic capacity of the pipeline.

Another procedure which may often prove to be effective and economical is to take advantage of passive soil pressures at the sides of the pipe which develop in response to horizontal movement against the soil as the pipe deforms, as described earlier. If the damage to the pipe is not too extensive, it may deform to a state of equilibrium wherein passive pressures build up sufficiently to prevent further deformation. This state can be determined by measuring the horizontal and vertical diameters at a number of places, marking the points between which the measurements are made, then repeating such measurements at weekly or monthly intervals. When a state of equilibrium is indicated, cracks can be reamed out with an air chisel, damaged concrete removed, and the areas patched with gunite or a suitable epoxy cement. Patches of this kind will not add to the strength of the pipe, but will protect the steel from corrosion. Two types of damage for which this procedure may be appropriate are shown in Figures 4 and 5.

Pipes which are more extensively damaged may be strengthened by drilling holes through the pipe walls at approximately the lower quarter points and injecting grout under pressure between the pipe and the bedding and backfill soil. This effectively increases lateral pressure and improves the bedding to such an extent that the supporting strength of the pipe is made adequate to carry the vertical load without further deformation. After this operation the loose and shattered concrete may be removed and a protective cover applied over the steel, as in the preceding paragraph. This method of repair was employed in the case of the damaged 108-in. pipeline shown in Figure 6. The repaired structure has served satisfactorily for nearly 18 years since repairs were made, and gives promise of much longer service. Figure 8 shows an 84-in. pipeline in which the bedding was improved by pressure grouting, after which several pipe sections were removed for observation of the distribution of grout. The balance of the pipeline was repaired by grouting and guniting and has served satisfactorily for about 16 years.

REFERENCES

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3. Spangler, Merlin G. Soil Engineering, 2nd Ed. International Textbook Co., 1960.

Review of Structural Design Methods for Aluminum Alloy Corrugated Culverts

A. H. KOEPF, Koepf & Lange, Consulting Engineers, Orinda, California

•A NUMBER of theories of structural design have been proposed during the last few years. This review represents a consolidation of these separate approaches into a framework from which a design method for aluminum alloy culverts may be developed. This review is limited to maximum fill height considerations and round pipe.

Flexible culvert should be analyzed in the same manner as any other engineered product. The past difficulty in absolutely defining the design limits has been due principally to the fact that the confining medium, soil, is nonhomogeneous and all too often unevenly compacted in backfilling, and thus indeterminate in value. Fill heights based upon analysis of support strength of culvert must consider the condition of the soil at the time of installation. As a general rule, once the culvert is installed, the soil, even if poorly compacted, will in time consolidate and become rigid with respect to the culvert. The culvert will then unload, reducing its support strength needs.

It should be noted that the behavior of the soil environment is a major factor in flexible culvert design. All too often theories may be proposed which make initial assumptions of soil behavior and proceed from there; in so doing the value of the analysis may be negated from the onset. The development of probable pressure and force distribution must be considered the key to accurate design analysis. It is because of the difficulty in determining soil loads that several theories of differing results may be given undue credence as the only method of design. This problem emphasizes the weight that must be given to engineering judgment in final fill height selection.

LOAD

It is generally recognized that loadings derived from Marston (2) represent as accurate a basis of design as can be attained by soils over and around culverts. The analysis of fill heights will be based on the mean condition, that of the full vertical wedge weight of the soil acting vertically and uniformly across the top of the culvert; thus,

$$W_c = \rho DH \tag{1}$$

where

W_c = weight on culvert, lb/ft;
ρ = density of soil, lb/ft^3;
D = culvert diameter, ft; and
H = fill height, ft.

SOIL

Next, the support strength of the soil should be established. This has been, and will continue to be, the indeterminate factor in design. The level of support capacity is determined by soil structure and compaction. Soils which are granular and easily compacted have excellent support strength levels. Soils which are heavy in clay or silt

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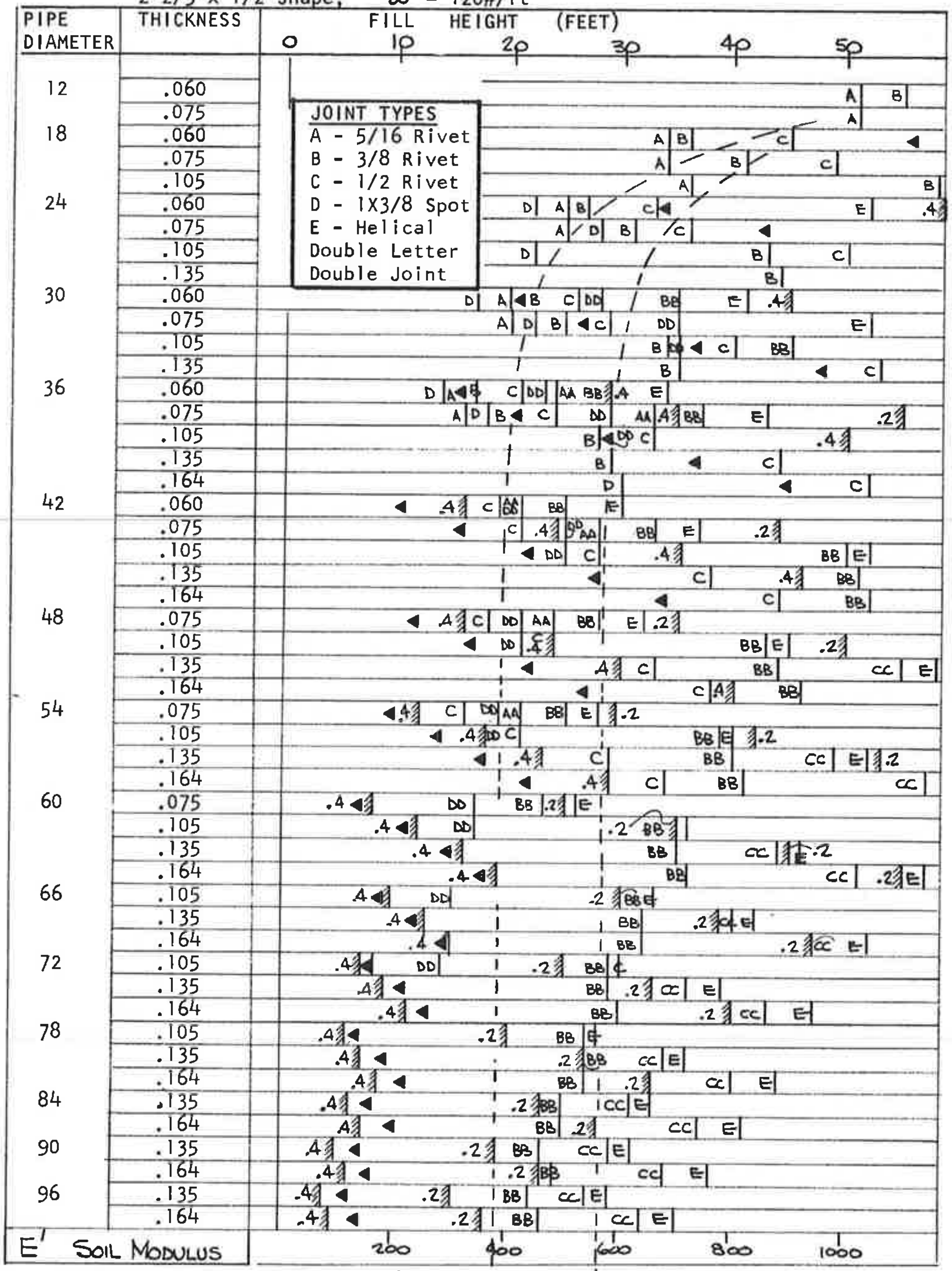
Note: This report originally contained a number of pages (Appendixes) of fill height calculations. These calculations may be obtained by writing directly to the author.

TABLE 1

COMPOSITE OF FILL HEIGHTS

Seam Strength, Ring Buckling, Total Stress, Deflection
2-2/3 x 1/2 Shape, $\omega = 120\#/ft^3$



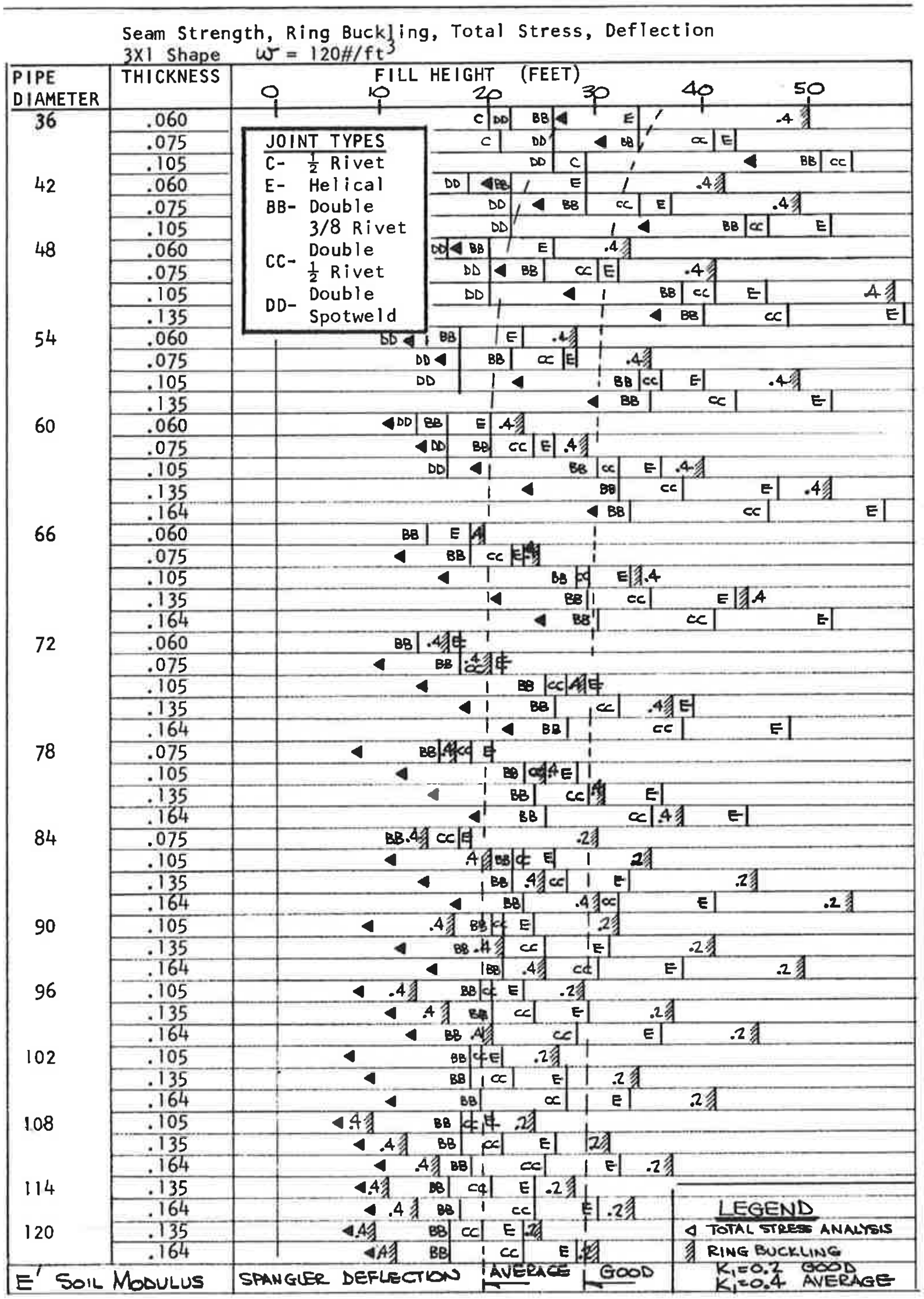
LEGEND --- SPANGLER DEFLECTION
AVERAGE | GOOD

◄ TOTAL STRESS ANALYSIS FROM MODIFIED SPANGLER

RING BUCKLING K_1=0.2 GOOD, K_1=0.4 AVERAGE

TABLE 2

COMPOSITE OF FILL HEIGHTS



content are low in support strength and difficult to compact. Significant movement under load may be anticipated for these poor structural soils. Design must be based on presumed levels of compaction, bearing in mind that the few flexible culverts which fail do so as a result of poor compaction or installation practices.

The exact level of support resistance, a combination of support strength of soil and degree of compaction, can only be approximated. Because of this limitation each theory can only be an approximation. Unfortunately, the problem of approximation is compounded, as a small change in external pressure distribution produces large changes in analytical results. It would seem that considerable restraint would have to be put on the blanket use of each theory.

DESIGN

Once a loading is established, design of the culvert should follow that of any other structure. It must be reviewed in thrust, bending, shear, deflection and instability. From these the ultimate support strength of the system may be determined. Safety factors would then reduce the solution to working levels. A series of design theories are given in Tables 1 and 2.

COMPRESSION RING

Thrust design is approximated by the compression ring theory (4). This approach presumes good compacted soil developing a uniform pressure around the periphery of the culvert and assumes the soil to be inelastic so that any shape will be rigidly maintained. With this assumption of soil behavior, it can be shown that the culvert will act as a ring in compression. The value of this approach is only as good as the assumption of uniform radial pressure from the soil. The hoop compression resists the pressure of the vertical load, therefore

$$2F = W_c \tag{2}$$

where

F = seam load, lb/ft; and
W_c = vertical load, lb/ft.

Design is based upon calculated or tested seam strengths with a safety factor of 3.0 and soil density of 120 lb/ft^3.

Coupon test data have been prepared for aluminum alloy culvert pipe for this analysis and are included and summarized in the Appendix. All types of seams, riveted, spot-welded, and helical lock seam, have been considered. Recent unpublished data indicate that stresses approaching yield strength of the metal may be used for helical culvert seam design. The composite fill heights in Tables 1 and 2 are prepared with the following code:

Joint Type	Description
A	Single row $^5/_{16}$-in. diameter rivets (this is standard for 0.060 and 0.075-in. sheet to 36-in. diameter, AASHO M 196-62I).
B	Single row $^3/_8$-in. diameter rivets (this is standard for 0.105 in. and thicker sheet to 36-in. diameter, AASHO M 196-62I).
C	Single row $^1/_2$-in. diameter rivets.
D	Single row spot welds 1 by $^3/_8$-in. oblong shape (reference AASHO M 209-63I).
E	Helical lock seam (reference AASHO M 197-62I).
AA	Double row $^5/_{16}$-in. diameter rivets (this is standard for 0.060 and 0.075-in. sheet 42-in. diameter and greater, AASHO M 196-62I).

Joint Type	Description
BB	Double row $^3/_8$-in. diameter rivets (this is standard for 0.105 in. and thicker sheet 42-in. diameter and greater, AASHO M 196-62I).
CC	Double row $^1/_2$-in. diameter rivets.
DD	Double row spot welds 1 by $^3/_8$-in. oblong shape (reference AASHO M 209-63I).

DEFLECTION

A second approach to design is that of deflection analysis by Spangler (2). This method considers uniform pressure across the plane of the top of the culvert, uniform pressure resistance across the plane of the invert, and horizontal side pressures as a function of the lateral displacement. When these loads are applied to the ring and solved for deflection the equation is

$$\Delta x = \frac{KW_c r^3}{EI + 0.061 E' r^3} \tag{3}$$

where

Δx = deflection of the culvert under load, in.;
K = bedding constant;
W_c = load on culvert, lb/in.;
r = radius of ring, in.;
E = modulus of elasticity of metal, lb/in.^{2};
I = moment of inertia of culvert, in.^{4}; and
E = modulus of soil reaction, lb/in.^{2}.

Design levels were established by limiting the solution to a deflection of 5 percent of the diameter of the culvert. The 5 percent value is limited to deflection under the applied load. Design values are given in Tables 1 and 2.

BENDING STRESSES

The pressure distribution outlined by Spangler (2) may also be considered as a method of evaluation of total bending and axial stresses of the ring under the applied load. For this purpose, the pressure distribution by Spangler was modified to allow for pressure variation across the top and invert (3). Bending and axial stresses under load may be determined from

$$S_{max} = \frac{Mc}{I} + \frac{R}{a} \tag{4}$$

where

I, c, a = properties of the culvert; and
M, R = moment and thrust at the crown as a result of soil forces.

Taking a design stress of 16,000 psi for aluminum alloy culvert (yield strength/1.5) the design fill limits are calculated (Tables 1 and 2). Fill heights which exceed the calculated values may be handled if the culvert is strutted or elongated during installation.

Using the stress analysis limit as applied to flexible culvert, soil reaction pressures and the modulus of soil reaction may be related as part of the analysis. From this, using a soil displacement level of 5 percent of the diameter, the modulus of soil reaction may be related to fill height:

$$E' = 20 H \tag{5}$$

Thus, at approximately 35-ft cover an E′ of 700 psi is attained, suggesting that where fills exceed this, special care is necessary to insure that the soil used is capable of developing soil reaction E′ levels greater than 700-plus adequate safety factor.

RING BUCKLING

Several papers have described a method of design using buckling concepts. A definite need exists to consider this aspect of design, and these approaches have been included in the review. Once again an original assumption of uniform pressure distribution is made, allowing for little or no moment to be developed in the ring. This limits the accuracy of this approach as it does in the compression ring.

Compression buckling (7) may be expressed in the column buckling or Euler form for all metals as a function of column slenderness, KL/r, where L is column length, r is radius of gyration of the culvert wall, and K is a fixity constant. (See Figs. 1 and 2.) Investigation shows that

$$\frac{KL}{r} = \frac{K_1 D}{r} \qquad (6)$$

where

D = culvert diameter, in.; and
K_1 = fixity of culvert wall with soil support.

Applying the theory of a fluid medium surrounding the culvert (8), a value of K_1 = 0.908 is established as the lower design limit and a value of K_1 = 0 is established for an inelastic medium as presumed by the compression ring theory. The true design values of K_1 lie between the two extremes.

Data presented by Meyerhof and Baikie (5) contained an excellent set of results which may be used to establish a level of K_1 for a condition of good granular compacted backfill.

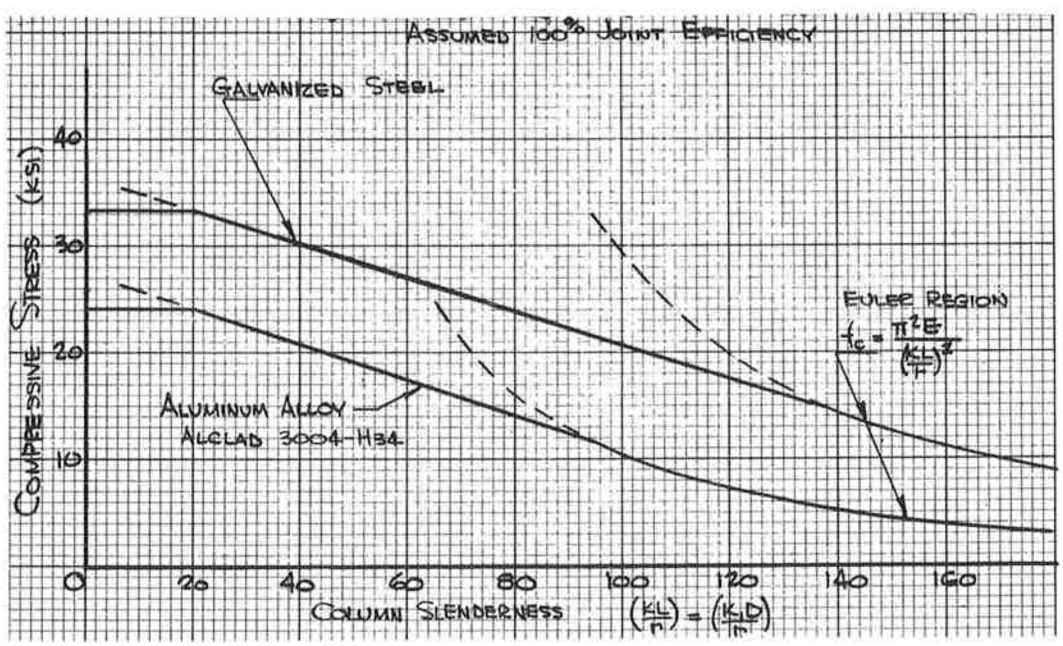


Figure 1. Curves of ultimate buckling stress, aluminum and galvanized steel culvert sheet.

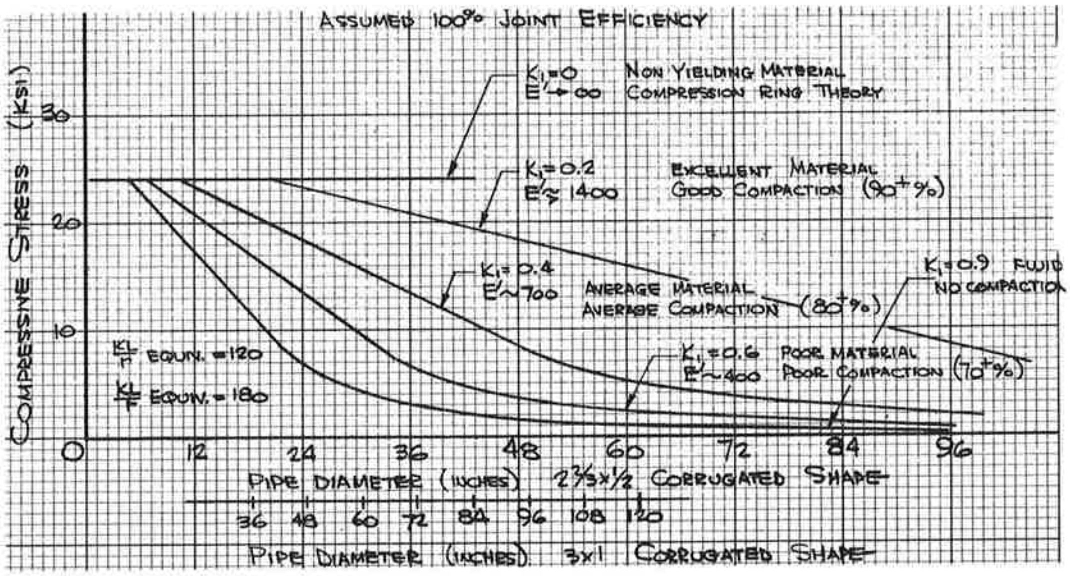


Figure 2. Effect of backfill material and compaction on compressive buckling stress, aluminum alloy, $2\frac{2}{3} \times \frac{1}{2}$-in. shape.

The data showed K_1 to be from 0.03 to 0.19 with soil modulus values of 1,530 to 12,950. From these data, design limits of K_1 of 0.2 and E′ of 1,400 were selected for good installation conditions. Design stresses and fill heights were developed from the column stress curves.

Watkins (6) gives a second set of data on ring buckling values. Using small tubes and controlled but normal conditions, values of K_1 in the range of 0.4 to 0.5 were determined. From this, and field experience, a level of K_1 of 0.4 and E′ of 700 were established as the basis of normal design. A value of K_1 of 0.6 is set for poor backfill conditions.

Once K_1 is established, KL/r becomes set and fill heights based on compression stresses are developed. A safety factor of 2.0 is used for this analysis with load from hoop compression. The K_1 of 0.2 is considered as a maximum limit and the K_1 of 0.4 the limit before requiring elongation or strutting (Tables 1 and 2).

FLEXIBILITY LIMIT

The ring buckling method established a means of approximating column slenderness ratios. This ratio may now be used as a means to define the flexibility of aluminum alloy culverts under load. In establishing this limit design, the average condition of K_1 of 0.4 is used.

$\frac{KL}{r}$	Culvert Flexibility Condition	Aluminum Alloy Culvert Diameter	
		$2\frac{2}{3} \times \frac{1}{2}$ in. Shape	3 × 1 in. Shape
120	Normal	≤54	<102
120-150	Flexible; elongation, strutting, or special care in handling in backfill required	60-66	>102
150-180	Very flexible; elongation, strutting, blocking, or special handling in backfill required	72-90	—

FLEXIBILITY FACTOR

The mathematical equation for an unsupported ring under external point loading has been proposed as a method of design control based upon flexibility. Such a method has no place in determination of fill heights, but is useful as a guide for relative handling flexibility in culvert placement. The equation for the loading deflection is

$$\Delta y = C \frac{W r^3}{EI} \tag{7}$$

where

Δy = deflection of culvert, in.;
W = load, lb;
r = radius of ring, in.;
E = modulus of elasticity, lb/in.^{2};
I = moment of inertia, in.^{4}; and
C = constant.

Considering a constant deflection ratio ($\Delta y/r$) and a unit load (W) the Flexibility Factor form is established as:

$$FF = \frac{D^2}{EI} \tag{8}$$

The limit levels suggested for steel are based on calculated values to include normal diameters, thicknesses, and corrugation shapes of existing products:

Shape	Steel Flexibility Factor
$2\frac{2}{3} \times \frac{1}{2}$ in.	4.33×10^{-2}
3×1 in.	3.33×10^{-2}
6×2 in.	2.00×10^{-2}

When the actual case of the unsupported ring (Eq. 7) is applied and a 5 percent deflection considered, the ring will be stressed far beyond the elastic limit of the metal. For steel, then, it is necessary to temper such a comparison of unsupported flexibility with an override of stress limitations. The values thus obtained are considerably more conservative than the limits proposed.

Recalculating the flexibility factor for steel with a stress limit of 20,000 psi, 5 percent deflection, or 500-lb/ft loading, the limits for steel must fall within either the deflection limit of D^2/EI of 7.60×10^{-2} or the stress limit of D/dt of 1,200, where d is depth of corrugation. This approach would result in much smaller diameters for a given thickness of sheet.

The deflection or stress limit analysis has a very different meaning for aluminum alloy culvert. When the initial conditions proposed are calculated, the aluminum has quite low stresses at the limits. Nonetheless, by applying the same analogy to aluminum alloy with a limit stress of 17,000 psi, the limit would be set by either D^2/EI of 7.60×10^{-2} or D/dt of 1,016. These values generally show that the lightest diameter thickness combination for aluminum alloy should be similar, a premise supported by considerable field experience.

3 × 1-IN. CORRUGATION

The commercial introduction of a 3 × 1-in. corrugated shape has been accompanied with several methods of fill height analysis of the kind reviewed previously. Calculations of fill height for 3 × 1 in. have been prepared from these theories. Joint coupons have been prepared and tested, and some pipe manufactured for structural review.

It remains to be seen where the 3 × 1-in. shape will fall in the fill height program for flexible culvert. The seam strength is no better than that of the $2^2/_3 \times {}^1/_2$-in. shape with equal fastening, and the flow friction factor is higher. However, because of the much higher wall stiffness, the 3 × 1-in. shape has advantages where bending, buckling, deflection, or instability may be considered to limit design, such as with large culverts or poor backfill material. In the larger culverts, the improvement is so marked that the maximum diameter has been suggested for increase from 96 to 120 in. and there is a justifiable opportunity to reduce metal thicknesses against the ½-in. depth shape. This will result in reduced unit length costs and a gain in overall structural integrity for such culverts. Another strong advantage is that the need for handling aids at installation is minimized, resulting in better control of the finished installation.

SUMMARY

This review includes consideration of the various conventional methods of development of design fill heights for aluminum alloy culverts. The limits have been appraised in thrust, bending, deflection, buckling, and flexibility; each related to the assumed soil environment behavior. The data have been superimposed in Tables 1 and 2 for comparison, and from this, it is expected that fill heights may be proposed. These data are deemed sufficient to comply with the needs for good design practice at reasonable product cost to the highway industry.

The review also points out repeatedly that more knowledge of soil pressures and forces and soil distortion is necessary before more accurate design data can be made available. It is understood that some of the research is contemplated.

Similarly, a computer program now exists that would allow the treatment of soil behavior as a simulated series of equivalent spring loads. This provides a method of accurately relating ring stresses and deflection to variations in external force changes. When better knowledge of soil behavior is coupled with such a program, more accurate representation of the system will be within reach, and the results might improve on this review.

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Appendix

TABULATION OF RESULTS

ALUMINUM CULVERT COUPON TESTS

Specimen Shape $2^2/_3 \times {}^1/_2$	Thickness	Rivets	DIA	Test Load 4-Rivet Pitch	Seam Failure Load (K/ft)	Load/Rivet Spot
1172-7	060	S	$^5/_{16}$	8,320	9.37	2,080
9	060	S	$^5/_{16}$	8,890	10.01	2,220
6281-16	060	S	$^3/_8$	8,950	10.08	2,240
17	060	S	$^3/_8$	9,500	10.70	2,370
18	060	S	$^3/_8$	9,450	10.65	2,360
5413-C1	060	S	$^3/_8$	9,500	10.70	2,370
C1	060	S	$^3/_8$	9,250	10.42	2,310
C1	060	S	$^3/_8$	8,900	10.20	2,230
See last page	060	D	$^5/_{16}$			
6281-31	060	D	$^3/_8$	14,500	16.33	1,810
32	060	D	$^3/_8$	14,750	16.61	1,840
33	060	D	$^3/_8$	14,900	16.80	1,860
			Spot			
6281-1 W	060	S	$1 \times {}^3/_8$	7,750	8.74	1,930
2 W	060	S	$1 \times {}^3/_8$	6,900	7.78	1,725
3 W	060	S	$1 \times {}^3/_8$	8,000	9.01	2,000
1172-15	075	S	$^5/_{16}$	9,050	10.19	2,260
3	075	S	$^5/_{16}$	8,200	9.24	2,050
4748-C2	075	S	$^5/_{16}$	9,000	10.13	2,250
C2	075	S	$^5/_{16}$	9,850	11.00	2,460
C2	075	S	$^5/_{16}$	10,300	11.60	2,570
5413-C2	075	S	$^3/_8$	10,100	11.38	2,520
C2	075	S	$^3/_8$	12,000	13.52	3,000
C2	075	S	$^3/_8$	11,500	12.96	2,980
6281-19	075	S	$^3/_8$	13,700	15.43	3,430
20	075	S	$^3/_8$	14,300	16.11	3,570
21	075	S	$^3/_8$	12,000	13.52	3,000
1172-33	070	D	$^5/_{16}$	18,750	21.13	2,340
35	075	D	$^5/_{16}$	17,500	19.72	2,190
41	075	D	$^5/_{16}$	16,000	18.02	2,000
4748-D1	075	D	$^5/_{16}$	17,000	19.17	2,120
D1	075	D	$^5/_{16}$	18,250	20.60	2,280
DD2	075	D	$^5/_{16}$	18,100	20.40	2,260
6281-34	075	D	$^3/_8$	20,900	23.55	2,610
35	075	D	$^3/_8$	20,000	22.55	2,500
36	075	D	$^3/_8$	20,300	22.85	2,540
46	075	D	$^3/_8$	19,000	21.40	2,370
47	075	D	$^3/_8$	19,000	21.40	2,370
48	075	D	$^3/_8$	19,500	22.00	2,440
5413-D2	075	D	$^3/_8$	18,350	20.65	2,290
D2	075	D	$^3/_8$	19,900	22.61	2,490
Shape						
6281-E1 1×3	075	D	$^3/_8$	19,800	19.80	2,480

Specimen Shape $2^2/_3 \times {}^1/_2$		Thickness	Rivets	DIA	Test Load 4-Rivet Pitch	Seam Failure Load (K/ft)	Load/Rivet Spot
	Shape						
6281-E1	1 × 3	075	D	$^3/_8$	21,050	21.05	2,630
E1	1 × 3	075	D	$^3/_8$	24,000	24.00	3,000
				Spot			
6281-4 W		075	S	1 × $^3/_8$	9,250	10.41	2,310
5 W		075	S	1 × $^3/_8$	9,850	11.10	2,460
6 W		075	S	1 × $^3/_8$	11,100	12.50	2,770
	Shape						
4748-49	1 × 3	075	D	$^1/_2$	26,000	26.00	3,250
50	1 × 3	075	D	$^1/_2$	25,000	25.00	3,130
51	1 × 3	075	D	$^1/_2$	22,000	22.00	2,750
1172-17		105	S	$^3/_8$	18,800	21.20	4,700
11		105	S	$^3/_8$	17,350	19.55	4,340
6281-22		105	S	$^3/_8$	19,650	22.15	4,910
23		105	S	$^3/_8$	19,800	22.30	4,950
24		105	S	$^3/_8$	19,500	22.00	4,870
1172-27		105	D	$^3/_8$	36,100	40.70	4,510
25		105	D	$^3/_8$	37,200	41.90	4,650
6281-37		105	D	$^1/_2$	30,600	34.45	3,730
38		105	D	$^1/_2$	31,100	35.00	3,880
39		105	D	$^1/_2$	32,300	36.40	4,030
				Spot			
6281-7 W		105	S	1 × $^3/_8$	8,800	9.91	2,200
8 W		105	S	1 × $^3/_8$	6,650	7.50	1,660
9 W		105	S	1 × $^3/_8$	8,400	9.46	2,100
6281-10		105	D	1 × $^3/_8$	17,700	19.92	2,210
11		105	D	1 × $^3/_8$	16,200	18.25	2,020
12		105	D	1 × $^3/_8$	14,750	16.62	1,850
1172-1		135	S	$^3/_8$	14,250	16.05	3,560
13		135	S	$^3/_8$	14,250	16.05	3,560
6281-25		135	S	$^3/_8$	29,000	32.70	7,250
26		135	S	$^3/_8$	26,350	29.70	6,590
27		135	S	$^3/_8$	29,000	32.70	7,250
4748-1		135	S	$^1/_2$	21,950	24.75	5,490
1172-39		135	D	$^3/_8$	31,200	35.20	3,900
31		135	D	$^3/_8$	29,300	33.00	3,660
6281-40		135	D	$^1/_2$	34,750	39.15	4,350
41		135	D	$^1/_2$	40,300	45.40	5,050
42		135	D	$^1/_2$	37,000	41.70	4,630
1659-1		164	S	$^3/_8$	14,900	16.80	3,720
9		164	S	$^3/_8$	15,950	17.98	3,990
6281-28		164	S	$^3/_8$	31,700	35.70	7,930
29		164	S	$^3/_8$	30,750	34.62	7,680
30		164	S	$^3/_8$	29,100	32.80	7,260
4748-1		164	S	$^1/_2$	28,050	31.60	7,000
2		164	S	$^1/_2$	24,900	28.05	6,230

Specimen Shape $2\frac{2}{3} \times \frac{1}{2}$	Thickness	Rivets	DIA	Test Load 4-Rivet Pitch	Seam Failure Load (K/ft)	Load / Rivet Spot
1659-3	164	D	$\frac{3}{8}$	36,350	40.90	4,540
4	164	D	$\frac{3}{8}$	35,850	40.40	4,480
5	164	D	$\frac{3}{8}$	30,700	34.60	3,940
6281-43	164	D	$\frac{1}{2}$	45,000	50.70	5,630
44	164	D	$\frac{1}{2}$	43,500	49.00	5,430
45	164	D	$\frac{1}{2}$	42,000	47.30	5,250
5413-D1	060	D	$\frac{5}{16}$	14,700	16.60	1,840
D1	060	D	$\frac{5}{16}$	12,100	13.62	1,510
D1	060	D	$\frac{5}{16}$	13,700	15.42	1,710
4748-DD1	060	D	$\frac{5}{16}$	12,100	13.62	1,510

COMMENTS ON COUPON TESTS

HALES LAB # 1172

1. 0.105" SPECIMENS TESTED EXTREMELY HIGH. THIS IS DUE TO AN IDEAL RELATIONSHIP BETWEEN SHEET AND RIVETS TO PRODUCE THE BEST INTERFACE FIT. DURING TEST CONSIDERABLE LOAD IS TAKEN BY FRICTION.

2. 0.135" DROPPED OFF AS THERE WAS MORE RIVET LOAD

HALES LAB # 1659

1. 0.164" WAS AFFECTED AS THE o.135" MATERIAL IN #1172 CAUSING AN APPARENT LEVENING OFF AT RIVET SHEAR LIMITS.

HALES LAB # 4748

1. $\frac{1}{2}$" RIVET COUPONS GIVE VERY LOW VALUES. THIS WAS DUE TO POOR AND INADEQUATE SHEET HOLDDOWN MEANS WHEN RIVETS WERE SET SO SHEET WAS ALLOWED TO FLOW OUT IN CONE SHAPE. THESE VALUES WERE NOT USED IN STRUCTURAL ANALYSIS.

2. 14 GA RIVET SIZE IN C2 INCORRECTLY IDENTIFIED. RIVETS WERE 5/16" DIAMETER.

3. 14 GA RIVET SIZE IN D1 & DD2 ARE CORRECT AT 5/16"

HALES LAB # 6281

1. BLIND RIVETS ARE EXPERIMENTAL FOR A VERSION OF NESTABLE CULVERT AND DO NOT APPLY IN THIS ANALYSIS. STANDARD RIVETS START AT #16.

2. SPOT WELD SPECIMENS HAD NOMINAL SIZE OF 1" LONG X 3/8" MAXIMUM WIDTH ON TOP ANVIL INDENTATION IN OVAL SHAPE AND REPRESENT A GOOD LEVEL SPOTWELD.

3. 1/2" RIVET SAMPLES ARE OF GOOD COMMERCIAL QUALITY AND HAVE BEEN INCLUDED AS A BASIS FOR DESIGN.

SEAM STRENGTH

ALUMINUM ALLOY CULVERT SHEET
BASED UPON LABORATORY TESTED COUPONS

Joint	Fastener		Sheet		Ultimate Test Strength (K/ft)	Design Seam Strength Compression, S. F. = 3.0
	Size	No. Rows	Shape	Thickness		
Rivet	5/16	1	2 2/3 × 1/2	.060	9.23	3.07
				.075	9.23	3.07
	3/8	1	2 2/3 × 1/2	.060	9.90	3.30
				.075	11.22	3.74
				.105	15.72	5.24
				.135	16.20	5.40
				.164	16.62	5.54
	1/2	1	2 2/3 × 1/2	.060	12.13	4.04
				.075	13.27	4.42
				.105	18.00	6.00
				.135	24.30	8.10
				.164	27.90	9.30
	5/16	2	2 2/3 × 1/2	.060	13.50	4.50
				.075	18.00	6.00
	3/8	2	2 2/3 × 1/2	.060	16.20	5.40
				.075	20.70	6.90
				.105	31.50	10.50
				.135	32.40	10.80
				.164	33.30	11.10
	1/2	2	2 2/3 × 1/2	.075	24.70	8.23
				.105	33.30	11.10
				.135	39.60	13.20
				.164	46.70	15.57
Spot weld	1 × 3/8	1	2 2/3 × 1/2	.060	7.86	2.62
				.075	10.33	3.44
				.105	8.10	2.70
	1 × 3/8	2	2 2/3 × 1/2	.060	13.50	4.50
				.075	16.20	5.40
				.105	16.20	5.40
Rivet	3/8	1	3 × 1	.060	8.80	2.93
				.075	10.00	3.33
				.105	14.00	4.66
	1/2	1	3 × 1	.060	10.80	3.60
				.075	11.80	3.93
				.105	16.00	5.33
				.135	21.60	7.20
				.164	24.80	8.27

SEAM STRENGTH

ALUMINUM ALLOY CULVERT SHEET

Joint	Fastener		Sheet		Ultimate Test Strength (K/ft)	Design Seam Strength Compression, S. F. = 3.0
	Size	No. Rows	Shape	Thickness		
Rivet	3/8	2	3 × 1	.060	14.40	4.80
				.075	18.40	6.13
				.105	28.00	9.33
				.135	28.80	9.60
				.164	29.60	9.87
	1/2	2	3 × 1	.060	19.20	6.40
				.075	22.00	7.34
				.105	29.60	9.87
				.135	35.10	11.70
				.164	41.50	13.85
Spot weld	1 × 3/8	1	3 × 1	.060	7.00	2.33
				.075	9.20	3.07
				.105	7.20	2.40
	1 × 3/8	2	3 × 1	.060	12.00	4.00
				.075	14.40	4.80
				.105	14.40	4.80
Helical seam or sheet	—	—	2 × 1/2	.060	18.95	6.31
				.075	23.70	7.90
				.105	33.20	11.05
				.135	42.70	14.22
				.164	51.90	17.30

Note: The safety factor of 3.0 is to be combined with a soil weight of 120 lb/ft^3 in calculation of fill height.

(a) Values based on individual fastener strength as follows:

Type	Size	No. Rows	Thickness of Sheet				
			.060	.075	.105	.135	.164
Rivet	5/16	1	2050	2050	—	—	—
		2	1500	2000	—	—	—
	3/8	1	2200	2500	3500	3600	3700
		2	1800	2300	3500	3600	3700
	1/2	1	2700	2950	4000	5400	6200
		2	2400	2750	3700	4400	5200
Spot weld	1 × 3/8	1	1750	2300	1800	—	—
		2	1500	1800	1800	—	—