HIGHWAY RESEARCH RECORD

Number 320

Joint Sealants, Paint and Pipe

6 Reports

Subject Areas

27	Bridge Design
34	General Materials
40	Maintenance, General

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Foreword

The six papers in this RECORD are devoted to joint sealants, paint, and pipe and will be of special interest to highway materials, bridge, and maintenance engineers.

Cook, in his paper on the photoelastic stress analysis of a preformed compression seal, shows that the photoelastic method of stress analysis is well suited for studying the stresses in preformed compression joint seals. A sample problem is given to illustrate the method, with a typical chevron seal shape chosen for analysis. Photographs of photoelastic stresses show the points of stress concentration and the magnitude of the stresses. An appendix includes photographs of photoelastic stresses in other joint seal configurations.

Gunderson presents a study of bridge joint seals placed on California bridges. Inspection and evaluation of these seals show that seals of polyurethane and neoprene have shown the best results of all materials. The cast-in-place polyurethane seals, if properly installed, will effectively seal joints having up to ½ in. of movement. Some seal failures are the result of poor construction practices. A movement rating system, which determines the movement capability, has been developed for preformed elastomeric joint seals.

The paper by Watson indicates the feasibility of solving problems at bridge joints by a systems approach. Marked similarities in performance requirements, insofar as a sealing system is concerned, are present no matter what structural type of bridge is concerned. Sealing systems based on the compression principle seem to offer the greatly increased performance levels necessary to the new structural sophistication. This discussion is intended to better acquaint the bridge designer with what is being done today in modular compression sealing systems. Capabilities, problems incurred, and construction practices are discussed.

Dzimian's paper discusses hot-poured sealants, which are the lowest in cost and most widely used sealing materials available today. Hot-poured sealants offer many advantages that other sealants do not, including low cost, ease of application, deeper penetration, conformity to any shape, no requirement for special equipment, and utilization of unskilled labor. Research is suggested that should result in a better understanding and improved quality of hot-poured sealants.

Rooney and his associates report on a California Division of Highways 16-year investigation of a number of paint systems for the protection of steel from corrosion in an aggressive marine environment. Evaluation of test sections of two coastal bridges showed that the best system after a 10-year exposure was an inorganic post-cure zinc-pigmented sodium silicate primer having a vinyl finish coat. All other systems provided definitely inferior protection, the next best being an all-vinyl type.

Heger and his associates report on a research project undertaken to evaluate the structural behavior of machine-made concrete pipe reinforced with welded wire fabric reinforcement and to determine the validity of previously developed design methods for this type of pipe. Results indicate significantly greater variability of both 0.01-in. crack strength and ultimate strength for machine-made pipe compared to previously tested cast pipe. As long as this possibility of greater strength variability is recognized, the design formulas previously developed for 0.01-in. crack strength and ultimate flexural strength of cast pipe also apply to machine-made pipe. The design formula for ultimate diagonal tension strength may also be applied to machine-made pipe, but only with a larger safety factor and with certain modifications of provisions tentatively suggested in an earlier report. Comparison of test results on companion pipe indicates that deformed wire fabric offers higher 0.01-in. crack strength than smooth wire fabric.

—Robert A. Anderson and John Beaton

Contents

The Photoelastic Stress Analysis of a Preformed Compression Seal

JOHN P. COOK, Department of Civil Engineering, University of Cincinnati

This paper shows that the photoelastic method of stress analysis is well suited for studying the stresses in preformed compression joint seals. A sample problem is given to illustrate the method. A typical chevron seal shape is chosen for analysis. Photographs of photoelastic stress clearly show the points of stress concentration and the magnitude of the stresses. An Appendix includes photographs of the photoelastic stresses in other joint seal configurations.

•THE GROWTH within the past 10 years in the use of preformed compression seals for joints in highway pavements has been quite dramatic. In the overall sealing market, which includes new construction plus re-sealing, the compression seals rank second behind the hot-poured asphaltic sealants. However, the preformed seals are probably specified for more contracts for new highway construction than any other type of seal.

Advantages and disadvantages of the compression seals have been explained in great detail in other publications (1, 2, 3, 4), and thus great depth is not needed in this paper. Only a few advantages and disadvantages will be given here in order to set this paper in proper perspective.

Advantages of the preformed compression seal are that it

1. Does the best job of keeping incompressibles out of the joint,
2. Is easily installed,
3. Does not require extensive joint cleaning, and
4. Makes the best looking joint of any known seal.

Disadvantages of the preformed compression seal are that it

1. Contains unknown stress levels and stress distributions within the seal,
2. Has an extremely high cost,
3. Does not keep water out of the joint, and
4. Requires straight, firm joint walls in order to function.

It costs from five to ten times as much to seal a pavement joint with a compression seal as with a poured-in-place sealant. Consequently, the compression seal must have a long enough service life to amortize its high initial cost. Here lies the paradox of compression sealing. The seal must have a long service life, yet no one can predict this service life because the stress intensity and stress distribution within the seal are unknown. All that is known at the present time is that the compression seals do function. They do the best job of keeping incompressibles out of the pavement joint.

The intent of this paper is neither to sell nor condemn compression seals. The purpose here is simply to show that the photoelastic method can be used to determine the stresses in the seals, and to demonstrate the method with a typical seal configuration.

Paper sponsored by Committee on Sealants and Fillers for Joints and Cracks in Pavements and presented at the 49th Annual Meeting.

DESIGN OF THE SEAL

Several dozen seal cross sections are currently being used in highway pavements. Some of these designs are undoubtedly excellent, whereas others are probably very poor. Some typical cross sections are shown in Figure 1.

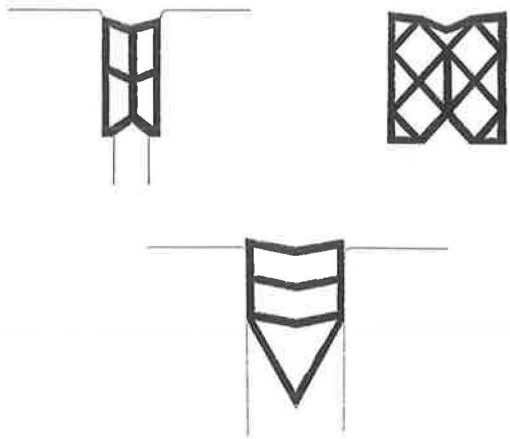

Figure 1. Typical seal cross sections.

The design of the compression seals has largely been a combination of engineering intuition, test data, and economics. The single paper by Dreher (5) has been the only published effort to provide a rational design basis for compression seals.

In most engineering design problems, a sequence is followed. Because stress is a function of load and shape (e.g., P/A), the loads are first determined and a shape is selected. Stresses are then determined. A selection of material and a possible modification of shape complete the design.

The designer of a compression seal, however, is forced to operate under a handicap. Loads, which are caused by the moving pavement, are largely unknown. Also, the shapes used for compression seals are too complex for conventional methods of stress analysis. Consequently, the designers have relied largely on test data. Interface pressure has become the accepted criterion of seal performance. Seal cross sections that have shown high laboratory test values of interface pressure have generally performed well in the field.

In the design or analysis of a compression seal, although loads are unknown, pavement movement can be calculated or measured. The advantage of the photoelastic method is that loads need not be known. Movements or deformations can be applied directly to the seal cross section and the stresses can be determined.

PHOTOELASTIC THEORY

Virtually every translucent material has two indexes of refraction when placed under stress. This property, called double refraction, is what makes photoelasticity work. When a ray of light enters a stressed model, it is broken down into two components, one corresponding to each index of refraction. This means very simply that one component takes longer to pass through the model than the other one does. Both components are retarded, or slowed down, to some extent as they pass through the model. Because the material is doubly refracting only under stress, it becomes apparent that the retardation of the two components is proportional to stress.

The problem, then, is how to measure the retardation. Ordinary light cannot be used because it vibrates in all planes simultaneously. Consequently, a ray of light is passed through a polarizing sheet that absorbs all components except those in a single plane. The light that emerges from the polarizer is vibrating in only one plane and forms a simple sine wave in this plane. The polarized light then enters the stressed model and is broken down into two components. As an example, let us orient the polarizer so that it transmits light in a vertical plane only. The two components of this light that emerge from the model are at some inclined angle to the vertical. As these two components emerge from the model, one is slightly behind the other because of the different indexes of refraction. A second polarizing sheet, called the analyzer, is then placed in the system, oriented at 90 deg to the polarizer. The analyzer then transmits only the horizontal components of the two light waves coming out of the model. Because one of the components is retarded more than the other one, the two sine waves emerging from the analyzer sometimes reinforce each other and sometimes cancel each other out. A viewer looking into the apparatus toward the light source then sees alternate bands of light and dark in the model. The number of bands, which are called fringes, is proportional to the retardation and, consequently, to the stress in the model.

An important aspect of photoelastic work is that the two components emerging from the stressed model correspond to the two principal stresses in the model. Photoelasticity has not been popular for structural design work because it measures only the relative retardation of the light waves and, consequently, only the difference between the principal stresses. This can be seen in the photoelastic formula

$$S_1 - S_2 = \frac{N\,fs}{t} \qquad (1)$$

where

S_1 and S_2 = the principal stresses,
N = number of fringes counted,
fs = a calibration constant, and
t = thickness of the model through which the light passes.

Under ordinary circumstances, the photoelastic formula does not give the value of either principal stress directly. However, if one of the principal stress values is equal to zero, the formula gives a direct value for the other principal stress. As an example, consider a simply supported beam with loads applied on top of it. The bottom surface of the beam is not loaded. Consequently, the stress normal to the bottom surface of the beam is equal to zero and the photoelastic formula yields a direct value for the stress along the bottom fibers of the beam. Stresses at interior points in the beam cannot be determined so simply. The formula yields only the stress difference, and supplemental techniques such as numerical integration must be used to determine individual stress values.

Stresses, then, can be determined directly at free, or unloaded, boundaries. This fact becomes of paramount importance in the analysis of a preformed compression seal. The seal is loaded only from the two sides. The top and bottom surfaces and all the interior reinforcing webs do not have any applied load and, consequently, are free boundaries. Certainly all the critical areas within the seal cross section are free boundaries and the stresses can be determined directly by simply counting the number of fringes in the stressed specimen.

SEAL CROSS SECTION USED

Seal cross sections are available in a variety of shapes, but the shapes most often used are the rectangular section and the chevron. The complete study from which these results are taken included the rectangular shape, the chevron, and one of the experimental shapes developed by Dreher (5). For purposes of simplicity, only the chevron shape is illustrated in this paper. Although the section chosen is modeled from an actual seal, no inference should be drawn about any company's product. The only purpose of this paper is to present a method of analysis. The Appendix shows pictures of the photoelastic stress patterns in the other shapes included in this study.

SPECIMEN PREPARATION

The specimens for this work were cast from various transparent resins. Molds for the specimens were shaped from a ½-in. thick solid polyethylene sheet. The resins used for casting were varied to suit the deformation required in the specimen. Four resins were used for various phases of the work: Solithane 113, a urethane from Thiokol Chemical Company; Epon 828, an epoxy from Shell Chemical Company; Epoxy No. 810 from Sika Chemical Company; and RTV 615, a silicone from General Electric. The photoelastic stress sensitivity of these clear materials varies widely. A deformation of less than 10 percent will cause four distinct stress fringes in a specimen of Solithane. A deformation of almost 100 percent is required to develop two stress fringes in the silicone. The most sensitive resin is not necessarily the best. Solithane, for instance, is excellent for measuring stresses when the stress values are low. However, at larger deformations, this resin shows so many fringes that they crowd together and the pattern becomes blurred and indistinct. Consequently, four resins with different sensitivities were calibrated for this analysis. A quick look back at Eq. 1 shows the effect of the sensitivity of the specimen. Stress is directly proportional to the number of fringes, N, and the sensitivity of the specimen, fs. A given level of stress can be maintained with a highly sensitive material and few fringes, or a less sensitive material and many fringes. The list below shows the calibration values for the four photoelastic materials.

Solithane 113	4 psi per fringe per inch
Epon 828	20 psi per fringe per inch
Sika Epoxy No. 810	12 psi per fringe per inch
RTV 615	40 psi per fringe per inch

ANALYSIS OF THE MODELS

There are two basic methods of counting the stress fringes in a photoelastic model. One method begins by locating a "source" or point of zero stress and counting fringes from this point. This source shows up as a black dot in the photoelastic pattern. The second method is simply to load the specimen slowly and count the number of fringes that pass a given point. Both methods are very well suited for compression seal analysis.

Before proceeding with the compression seal analysis, a sample problem will be worked to demonstrate the method.

Example: Determine the stress in the bottom fiber at midspan of a simply supported beam with a span of 4 in. and dimensions ¼-in. thick by ¾-in. deep, and a 22.5-lb load applied at midspan (Fig. 2).

Theoretical Solutions:

$$M = \frac{PL}{4} = \frac{22.5 \times 4}{4} = 22.5 \text{ in.-lb}$$

$$\text{Section modulus } (Z) = \frac{td^2}{6} = \frac{\frac{1}{4} \times (\frac{3}{4})^2}{6} = 0.0234 \text{ in.}^3$$

$$\text{Stress} = \frac{M}{Z} = \frac{22.5}{0.0234} = 960 \text{ psi}$$

Photoelastic Solution (Fig. 3):

Calibration constant, fs = 60 psi per fringe per inch
Number of fringes, N = 4 (counted from picture)
Beam model thickness = ¼ in.

$$S_1 - \overset{=0}{(S_2)} = \frac{N\,fs}{t} = \frac{4 \times 60}{\frac{1}{4}} = 960 \text{ psi}$$

Photoelastic stress analysis can be used to determine the points of maximum stress in a seal cross section and to determine the magnitude of the principal stresses, and is particularly valuable for comparing the efficiency of different cross sections. The selection chosen for analysis here is a 13/16-in. seal, which was scaled upward in model size for easier photographing.

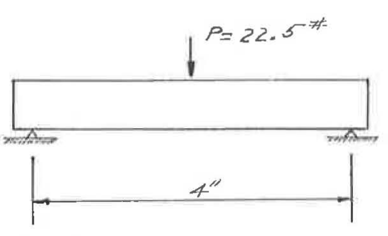

Figure 2. Simply supported beam with load at midspan.

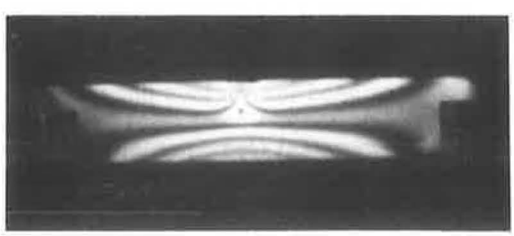

Figure 3. Photoelastic stress pattern in a simply supported beam.

Three stress (or deformation) levels are of interest in the proper functioning of a compression seal: (a) stress at time of installation, (b) stress at minimum joint width, and (c) stress at maximum joint width. All three of these areas have been investigated, but to simplify the presentation, only one level of deformation is shown here. The stress level at maximum joint width has been chosen for illustration for two reasons:

1. The seal must continue to exert an interface pressure when the joint is at maximum opening; and
2. The preformed sales are extruded from elastomers that have a nonlinear stress-strain relationship. By choosing a minimum value of seal deformation, the linearity of stress and strain can be safely assumed and the photoelastic method can be shown in its simplest form.

Deformations were applied to the seal by means of a simple loading jig that consists of two parallel plates. One plate is fixed; the other plate is moved by a simple thumb screw. Deformations were measured by a caliper mounted on the loading jig. The deformations shown in the following photography correspond to the $^{13}/_{16}$-in. seal compressed to $^{3}/_{4}$ in., or a deformation of 7.7 percent in the seal. Photographs of the stress fringes are shown at 3 and 7.7 percent. The sequence of photographs shows the points at which stress fringes first appear, which are critical points of stress in the seal. The sequence also shows how the number of fringes increases with increased deformation.

Figure 4 shows that the junction of the center webs is the point to watch with further deformation. Figure 5 can be used to determine the magnitude of the stresses. The black dot appearing at the center web junction is a "source" or point of zero stress. Counting black fringes upward and to the left from this dot shows four complete fringes with the fifth fringe just barely visible. In the photoelastic formula, then, N = 5. The stress at the junction of the interior web members can be determined directly from the

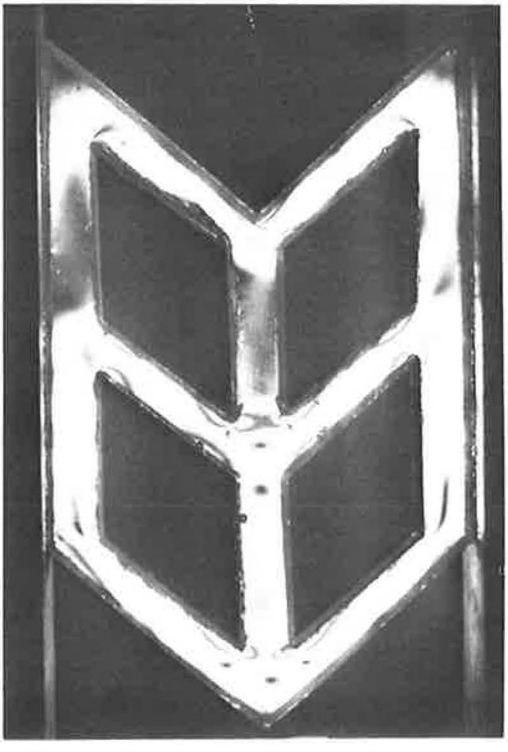

Figure 4. Chevron seal at 3 percent deformation.

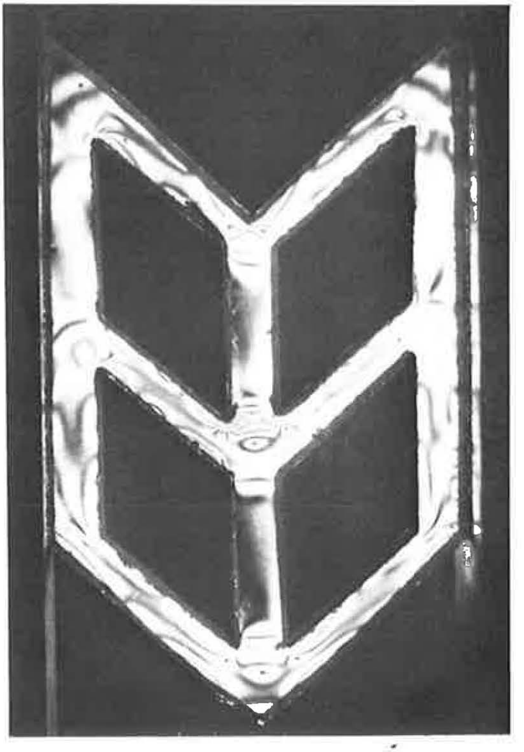

Figure 5. Chevron seal at 7.7 percent deformation.

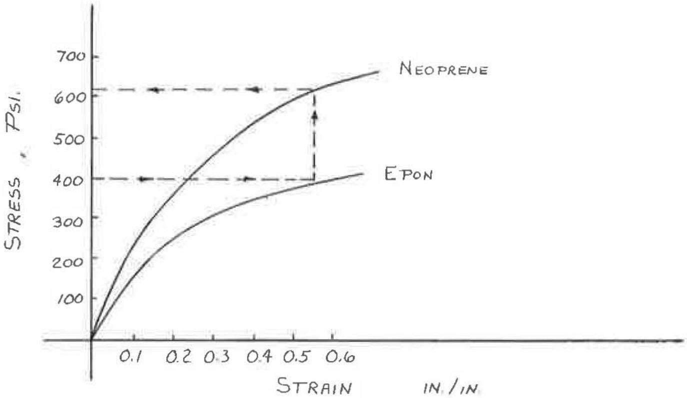

Figure 6. Modulus curves for seal and model materials.

photoelastic formula. The specimen is formed from the Sika Epoxy, which has a calibration value of 12 psi per fringe per inch. Model thickness is ½ in. Therefore,

$$S_1 - \overset{(=0)}{(S_2)} = \frac{N\,fs}{t} = \frac{5 \times 12}{1/2} = 120 \text{ psi}$$

Figure 5 shows that the top and bottom of the seal also have high stress concentrations at the center of the seal. At both the top and bottom locations, counting from the source to the edge, three fringes are seen clearly with the fourth fringe just beginning. Therefore, at these points N = 4. Stresses at these points are

$$S_1 - \overset{(=0)}{(S_2)} = \frac{N\,fs}{t} = \frac{4 \times 12}{1/2} = 96 \text{ psi}$$

In order to determine a definitive value of stress at large values of deformation, the nonlinearity of both the seal and model materials must be considered. Stress-strain curves must be plotted for both the seal and the model materials. Figure 6 shows this conversion, using the Shell Epon Resin, which has a calibration value of 20 psi per fringe. Compressing the seal specimen 50 percent gives 10 stress fringes at the junction of the center webs. The stress in the model is

$$S = \frac{10 \times 20}{1/2} = 400 \text{ psi}$$

Because strain is a function of load and shape, the strains in the model and the actual seal are equal. Consequently, enter the curves with the model stress of 400 psi and find the value of strain. At this value of strain read upward to intersect the neoprene curve and find the value of stress in the neoprene seal. The seal cross section shown in Figure 5, when extruded from neoprene, will have a stress of 610 psi at the junction of the center webs when compressed 50 percent.

RECOMMENDED RESEARCH

The analysis of preformed compression seals has only begun. It is to be hoped that further research may answer some of the following questions:

1. What is the effect of stress level on the life expectancy of various elastomers, such as neoprene and EPT?
2. What is the optimum relationship between stress and interface pressure in a compression seal?

3. What is the effect of stress relaxation on stress distribution and interface pressure?
4. Are the stresses in large modular seals linearly dependent on joint movement?

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Appendix

PHOTOGRAPHS OF PHOTOELASTIC STRESS PATTERNS IN OTHER JOINT SEAL CONFIGURATIONS

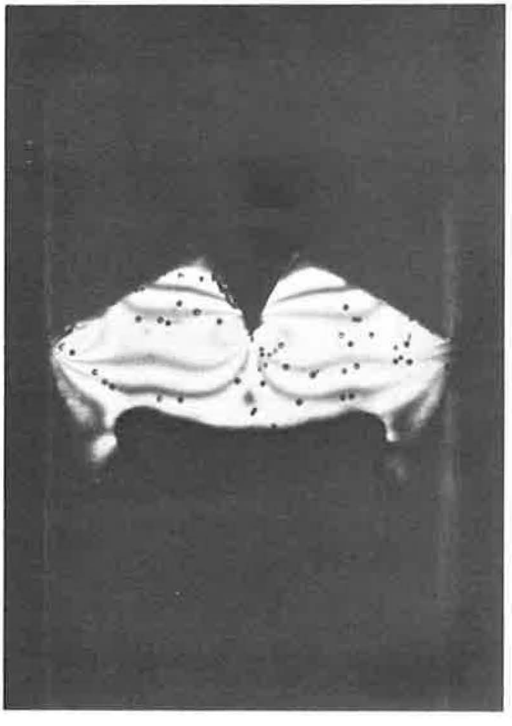

Figure 7. Experimental shape by Dreher: stress during installation.

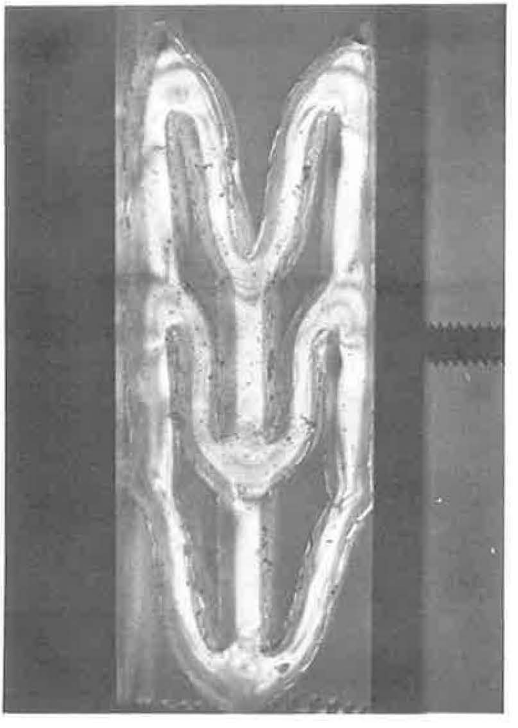

Figure 8. Chevron shape at 50 percent deformation.

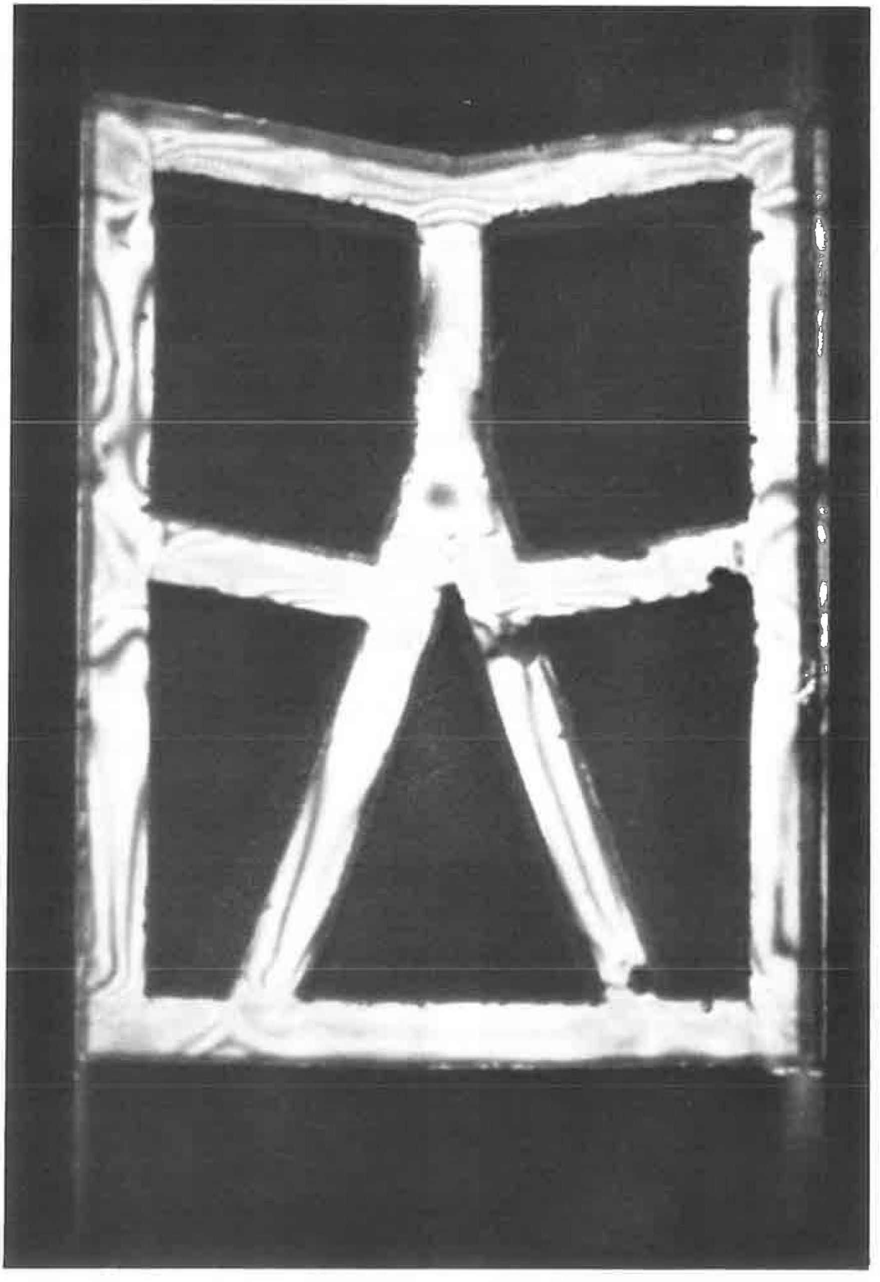

Figure 9. Rectangular shape at 8 percent deformation.

Bridge Expansion Joint Sealants

BRUCE J. GUNDERSON, California Division of Highways

Bridge joint seals placed in the field have been inspected and their effectiveness evaluated. Of the materials tested, seals of polyurethane and neoprene have shown the best results. The cast-in-place polyurethane seals, if properly installed, will effectively seal joints having up to ½ in. of movement. Poor construction practices account for a number of seal failures. Joints for preformed elastomeric seals should be engineered to fit the given conditions. A "movement-rating" system, which determines the movement capability, has been developed for preformed elastomeric joint seals.

•THE CALIFORNIA DIVISION OF HIGHWAYS has been studying joint sealants for the past 11 years. During that time many seals have been experimented with, including asphalt latexes, hot asphalts, silicone, polyvinyl chloride, nitrile rubber, butyl rubber, neoprene polymers, epoxies, coal tar extended polysulfides, modified polysulfides, straight polysulfides, polyurethanes (two-component and one-component), polyurethane foams (asphalt-impregnated, butyl-impregnated, neoprene-jacketed, plain), neoprene sheet, preformed elastomeric seals, neoprene header with steel reinforcement, and aluminum extrusions.

In the last few years, we have concentrated our research on three basic types of seals: two-component polyurethanes, polyurethane foams, and preformed elastomeric seals. The preformed elastomeric seals are now receiving the greatest emphasis.

CONSTRUCTION PRACTICES

During construction it is very difficult to get a properly formed joint to seal against. First of all, the concrete surface is usually porous or poor to bond to. Second, improper edging usually results in an irregular vertical surface. Third, if the joint groove is formed with wood, the stripping of this wood usually fractures the concrete edge (Fig. 1). Fourth, steel rollers damage the joint edges. Many of these fractures remain in an incipient failure stage until the sealer pulls them off or traffic breaks them off. A sizable number of what we call joint-seal failures are actually concrete spalls caused by construction practices. A good way to check for these hidden fractures is to drag a chain or tap a hammer along the joint. The incipient fractures will be readily apparent by a dull thud sound.

In patching joint spalls, another series of problems arises. The epoxy work has to be done carefully or the patches will fail. It is also more difficult to bond a seal to an epoxy surface. The two possible reasons for this are (a) the bond-breaking agent that is used on the forms, and (b) placement of the sealant before the epoxy has cured.

Armored joints would help alleviate many of the problems associated with formed joints, but there are also problems with armored joints. Some of these problems are loose or broken anchor straps, spalled concrete adjacent to the armor, and poor consolidation of concrete under the armor. Also, the riding characteristics of the roadway surface suffer from poor vertical alignment of the armor assembly.

California presently specifies a saw-cut joint groove (Fig. 2) in an attempt to minimize joint problems. This gives a uniform joint width. It also allows the selection of the joint groove width, with temperature taken into consideration, after the major

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Figure 1. Formed joint groove.

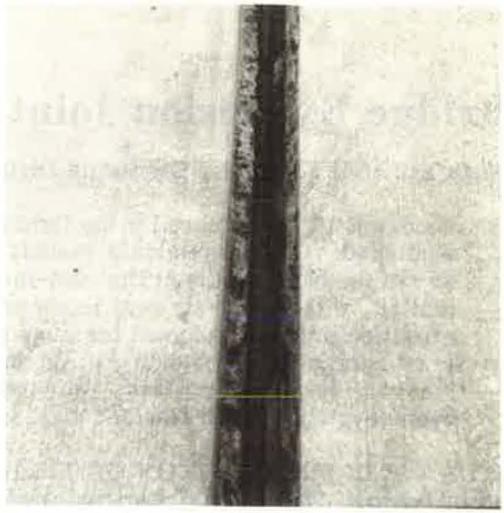


Figure 2. Saw-cut joint groove.

portion of the shrinkage, creep, and shortening of the structure have taken place (a very important feature for preformed elastomeric joint seals). It is important, however, to round or bevel the edges of the saw-cut groove. The saw-cutting will in some instances expose fractured or damaged concrete that would not otherwise show up until traffic was on the structure.

Another problem during construction is debris that sifts into the joints before they are sealed. Unless this debris is cleaned out prior to sealing the joint, future spalls may occur (Fig. 3). Normal practice in California is to clean out all debris, including the expansion joint filler, down to the waterstop, which is usually 6 in. below the deck surface, just prior to sealing the joint.

Incidentally, we do use a waterstop (Fig. 4) in conjunction with the joint seal in an effort to get a satisfactory seal. The material used is a polyvinyl chloride. Some of the difficulties with our present waterstop are as follows:

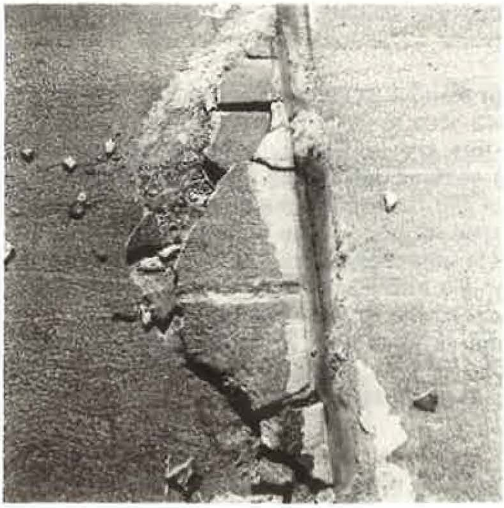


Figure 3. Joint spall.

1. The material stiffness varies with temperature.
2. The waterstop is difficult to place.
3. During concrete placement, it is difficult to keep the concrete from (a) flowing between top of bulb and expansion joint filler and (b) flowing between leg and expansion joint filler.

RESEARCH PROCEDURES

Experimental sealants are tested in our laboratory. If the results warrant, the seal is then placed in the field on an actual bridge to test its capabilities in use. The resident engineer submits a joint sealant report when the joint sealing is completed. Included in this report are contributory structure length to joint movement, type of structure, dimensions of formed joint, dimensions of joint when

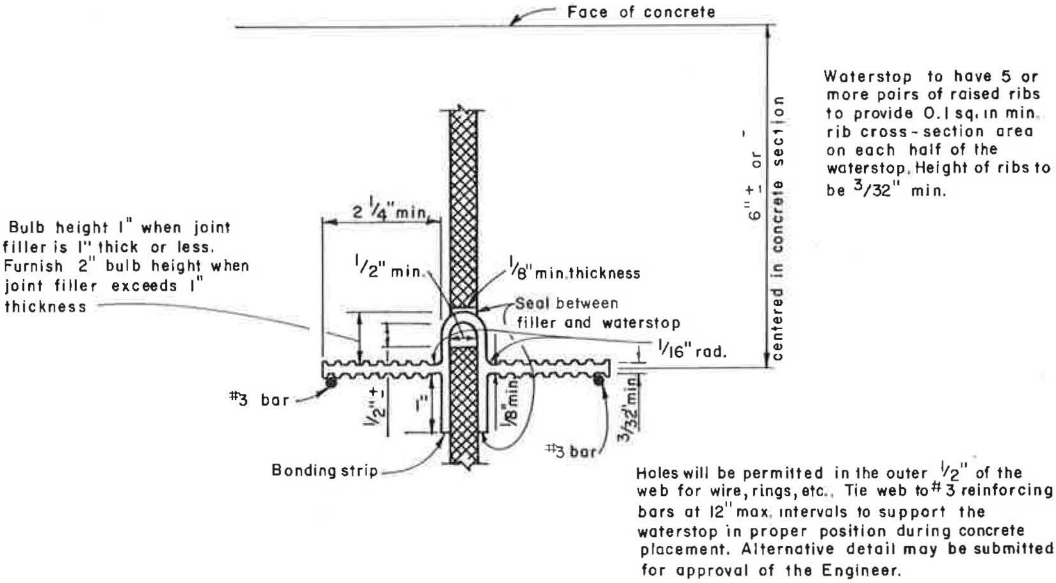


Figure 4. Waterstop detail.

sealed, joint sealant, type of primer or adhesive, date of installation, ambient temperature when sealed, weather conditions, total lineal feet, cost per lineal foot, and party installing the sealant.

On selected bridge deck joints, movement scribes (Fig. 5) are placed on the railing to measure the actual movement that the joint is subjected to over a period of time. The sealants are inspected periodically and a record is assembled and maintained.

RESEARCH RESULTS

Of all the material placed during this study, those from the polyurethane and neoprene families have shown the best results as effective bridge deck joint seals. Performance observations of seals placed in the field since 1966 are included in the Appendixes.

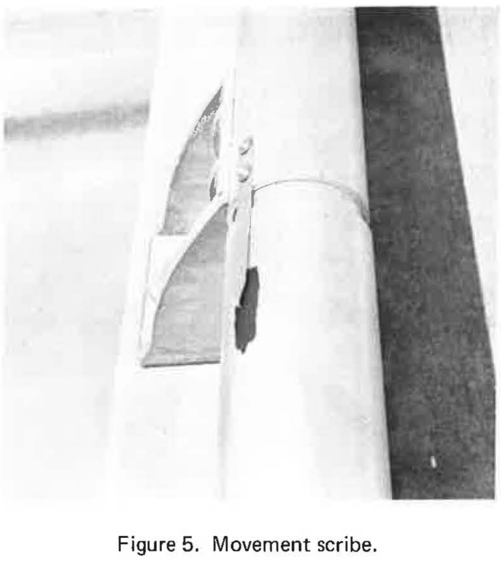


Figure 5. Movement scribe.

Polyurethane Cast-in-Place Seals

The polyurethane seal (Fig. 6) has its best chance for success if the joint movement is limited to ½ in. or less (Table 1). In joints having larger movement, the chances for a satisfactory seal diminish rapidly. The common type of failure is in adhesion to the concrete. Some of the more common installation difficulties experienced with this type of seal have been

1. Inadequate coverage of the joint face with primer,
2. Not allowing primer to dry sufficiently before sealing,
3. Inadequate mixing of the sealant,
4. Incorrect ratio of sealant components, and

TABLE 1

POLYURETHANE CAST-IN-PLACE SEALS

Material	Performance	Expected Movement (inches)												
		1/8	1/4	3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2	Over 1 1/2
PRC 3105	Satisfactory	4	8	4	2	1	1			1	1			
(machine grade)	Minor tears	1		4			1			1				
	Fail				1	5	1	1			4			2
PRC 3105	Satisfactory	1	2	3	3	2			1	1				
(pourable)	Minor tears			1	1	2	1				1			
	Fail						1	2		1	3	1		
Terraseal 100	Satisfactory	2	1	2	2		1				1		1	
(one-component)	Minor tears		1						1	1				1
	Fail	1						1				2		
Endoco U-Seal 3201	Satisfactory	1	1		4		1			2				
(machine grade)	Minor tears					1								
	Fail													1
Uralane 8305	Satisfactory			1	1									
Ureseal 200	Minor tears	1	1	1										
Ceelrite	Fail	1								1				
Sikaflex T-68	Satisfactory										1			
(pourable)	Minor tears													
	Fail		1											

Notes: 1. Condition of seals placed since 1966 as of inspection of June 1969.
2. Figures represent number of joint seal reports in each category.

5. Poor sealant shape factor (at present a width-to-depth ratio of 3 to 1 is used).

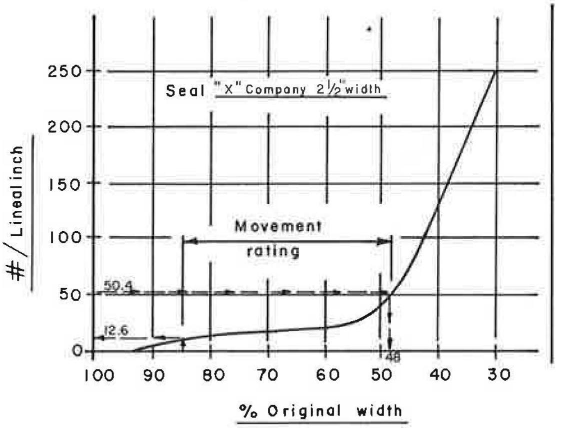
Preformed Elastomeric Seals

We have found that the size of preformed elastomeric seals (Fig. 7) must be chosen very carefully, and the joint geometry designed to fit the seal and the movement expected. We use compression seals in joints with up to 2 in. of movement.

A "movement-rating" system to determine the design movement capability of the seal has been developed for preformed elastomeric joint seals. The movement-rating

Figure 6. Polyurethane sealant.

Figure 7. Preformed elastomeric seal.



For example :

1. Pressure at 85 % nominal width = 12.6
2. 4 x pressure at 85 % nominal width = 4 x 12.6 = 50.4
3. % nominal width at 4 x pressure = 48 %
4. Movement Rating = nominal width (.85 — .48) = 2½" (.85 — .48) = 0.925"

Figure 8. Pressure-deflection curve for preformed elastomeric seal.

value is derived from the seal's pressure-deflection curve. The pressure at 85 percent of the uncompressed width is taken. This pressure is then multiplied by four. The movement rating of the seal is the deflection value between these two pressures. An example is shown in Figure 8.

This criterion, plus our rather arbitrary specification that the depth of seal shall be at least 75 percent of the nominal width, shall have stability of the top edges, and shall have 3 psi minimum pressure generation at 85 percent of nominal width, sums up our present method for selection of size and configuration.

Our success with preformed elastomeric seals dates back to our original installations of thick-wall seals in 1964. Even though these seals have been performing satisfactorily, they were very difficult to place, and were, in some cases, damaged during installation. In our larger moving joints (1½ to 2 in.), the size of the thick-wall seal required is excessive. With this in mind we are presently field-testing new thin-wall design seals. Some of the more common installation difficulties with the preformed elastomeric seal have been the following:

1. Top of material has been placed above deck level.
2. Leaks have occurred at changes in alignment.
3. Seals have been installed upside down or even sideways.
4. Adhesive has been wiped off the joint sides as the seal is slid in.
5. Sand intrusion has occurred because of poor or no adhesive.
6. Seal has been difficult to place in hot weather when the joint has closed up.
7. The maximum length available without splice has been 60 ft in some configurations, and satisfactory splices have been difficult to obtain.

Asphalt-Impregnated Polyurethane Foam

Asphalt-impregnated polyurethane foam (Fig. 9) is effective in sealing out solids in mild climates. It becomes very hard in cold climates and tends to lose its sealing capability.

Figure 9. Asphalt-impregnated polyurethane foam.

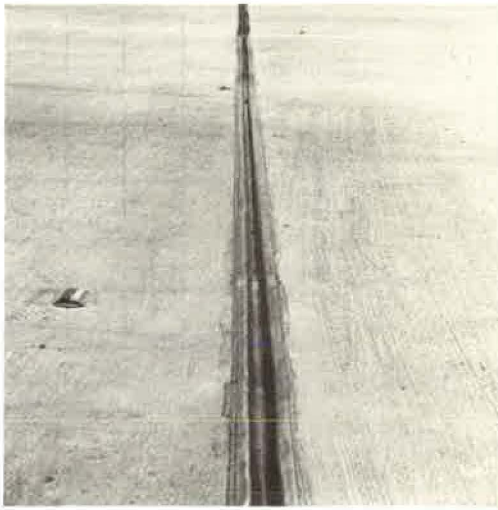


Figure 10. Neoprene strip.

Neoprene Sheet

Elastomeric sheets $\frac{1}{16}$-in. thick bonded to the deck concrete with a loop formed down into the joint were tried (Fig. 10). It was thought that this would be a fast and easy method for maintenance replacement of defective joint sealants. The neoprene did not maintain bond with the concrete, however, with traffic riding on the neoprene. This is an effective seal on elements not subjected to traffic.

Neoprene-Shielded Polyurethane Foam

Neoprene sheet bonded to polyurethane foam is a hybrid seal (Fig. 11). It combines the advantages of a thin-wall elastomeric seal with the inert qualities of plain polyurethane

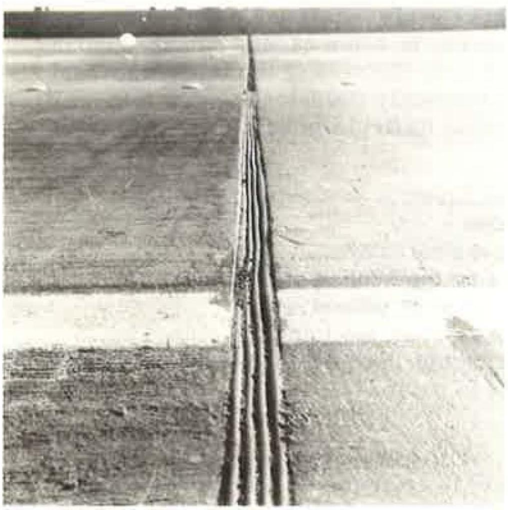


Figure 11. Neoprene-shielded polyurethane foam.

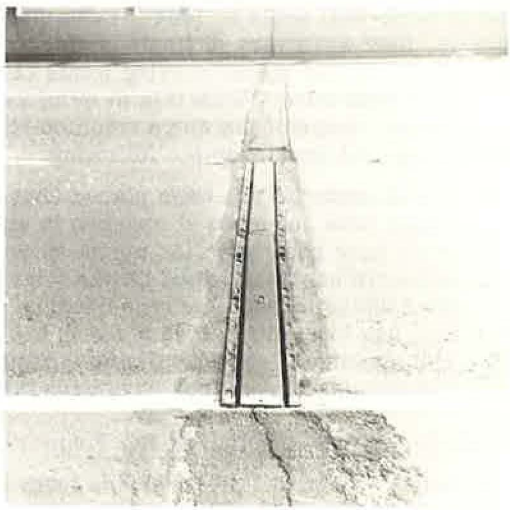


Figure 12. Transflex 150.

foam. The neoprene shields the foam from the detrimental effects of direct sunlight. The field installations, however, have had the following difficulties:

1. Cracking of the poor grade of neoprene used has occurred.
2. Inadequate bonding of the foam and neoprene has occurred.

We are presently field-testing extruded sections of this type of seal.

Transflex 200 (formerly Transflex 150)

This neoprene header with steel reinforcement functioned well for 2 years (Fig. 12). The plant-mix surfacing adjacent to the header, however, settled $\frac{1}{4}$ in. with the result that the neoprene edges of the header are now delaminating under the pounding of traffic. Additional installations of this type material are planned on concrete-surfaced decks.

CONCLUSIONS

1. Polyurethane poured-in-place seals should be restricted to movements of $\frac{1}{2}$ in. or less.
2. Preformed elastomeric joint seals, although far from the ideal, are the best seals we have at the present time.
3. The joint must be carefully engineered for compression joint seals.
4. Saw-cut joints are superior to formed joints.
5. Good inspection is a prerequisite for joint casting, preparation, and sealing.

FUTURE STUDIES AND RESEARCH

Much more knowledge is needed concerning

1. Adhesives for compression seals,
2. Pressure generation requirements for compression seals,
3. Sealing of skewed joints, and
4. Joint movement and temperature relationships.

Appendix A

CATALOG OF SEALANTS FIELD-TESTED TO DATE

I. Compression Seals
   A. Preformed elastomeric joint seals
      1. Acme S-497
      2. Acme B-496
      3. Acme B-462
      4. Acme S-500
      5. Brown B-2500
      6. Brown C-2500
      7. Brown D-3000
   B. Polyurethane foams
      1. Asphalt-impregnated
         a. Compriband
         b. Ureseal
      2. Neoprene-shielded
      3. Untreated
   C. Butyl rubber

II. Mechanical Seals
   A. Transflex 200
   B. Elastomeric sheets bonded to deck surface
      1. Neoprene
      2. Urethane

III. Poured-in-Place Seals
   A. Two-component polyurethane
      1. PRC 3105
      2. U-Seal 3201
      3. Ureseal 200
      4. PRC 3000
      5. PRC 220
      6. PRC 210
      7. Coast Pro Seal 962
      8. Allied
      9. Tabo
   B. One-component polyurethane
      1. Terraseal 100
      2. PRC RW-370-01
   C. Polysulfide
      1. Pressite 54, 55, 404, 1175.55
      2. Coast Pro Seal F-37
      3. Edoco 170281, 170282
      4. Fuller 400
      5. Churchhill 3C-51
      6. Chem-Seal
   D. Silicone
   E. Polyvinyl chloride
   F. Two-component neoprene
      1. Polymeric N-25-4-36
      2. Polymeric N-25-4-19
   G. Epoxy
      1. Epothak 2100
      2. Ceelrite
      3. Coast Pro Seal 805
      4. Epocast H 1356
   H. Asphalt latex

IV. Products to Be Evaluated in the Near Future
   A. Preformed elastomeric joint seals—new thin-wall cross sections
   B. Transflex 400
   C. Sikaflex T-68—two-component polyurethane
   D. Superseal 444—hot-poured polymer
   E. Uralane 8305—two-component polyurethane
   F. Extrusions of neoprene-covered polyurethane foam
   G. Aluminum extrusion

Appendix B

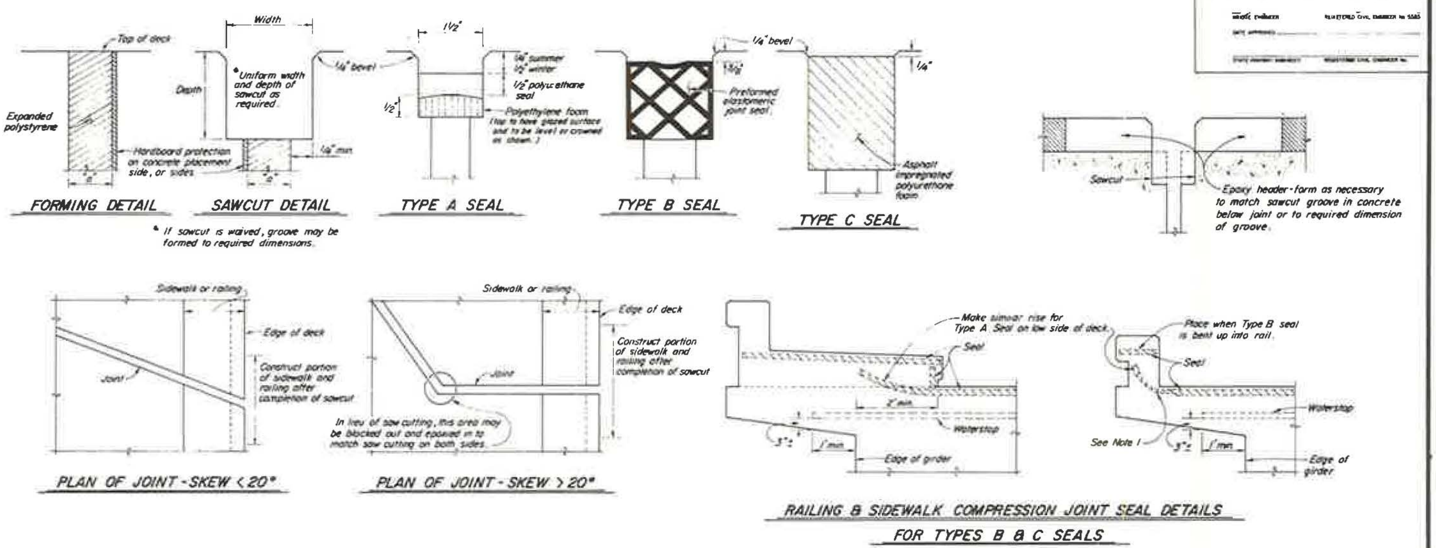
COMPRESSION SEAL INSTALLATIONS

Seal	Satisfactorily Sealed	Installation			Sand Intrusion		Cracking	Extruded From Joint
		Too High	Damaged	N.G.	Minor	Major		
S-497	4	1			3			
S-500				1	1			1
B-496	1	1						
B-610	2				1			
B-462		1						
B-2500		2			1	1		
C-2500	2		1					
D-3000	1							
Polyurethane foam:								
Asphalt-impregnated	6				5	7		
Neoprene-jacketed	1	3			1	2	6	
Open cell (not a water seal)	3				1	2		

Notes: 1. Inspection as of 6/69.
2. Figures represent number of joint seal reports in each category.

Appendix C

TENTATIVE JOINT SEAL STANDARD PLAN



DIMENSIONS "a" OF JOINT REQUIRED

Movement Rating (M.R.)	Bridge Type	"a" Dimension Deck Concrete Placed: Winter	Fall-Spring	Summer
2"	All except CIP/PS	2"	1½"	¾"
	CIP/PS	1½"	1"	½"
1½"	All except CIP/PS	1½"	1"	½"
	CIP/PS	1¼"	¾"	¼"
1"	All except CIP/PS	1"	¾"	½"
	CIP/PS	¾"	½"	¼"
½"	All except CIP/PS	¾"	¾"	½"
	CIP/PS	½"	¼"	¼"

NOTES:

1. Make cut in seal as shown, to bend seal on low side of deck if required. Do not bend seal on high side of deck.
2. Opening in rail and sidewalk to be constructed to the exact existing width of sawn deck joint.
3. Conduit in rail shall be placed to avoid interference with seal.
4. Sawcut groove widths shall be as ordered by the Engineer.
5. Waterstop shall have to be lowered for joints with a Movement Rating of 2" to provide ..... clearance.
6. Depth of sawcut as required by joint seal manufacturer.

Tentatively scheduled
to replace standard plan
dated 11/68.

6 /69 — XS-12-45

JOINT SEAL DETAILS

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Appendix D

APPROXIMATE MOVEMENT RATINGS AND GROOVE WIDTH SELECTION FOR PREFORMED ELASTOMERIC JOINT SEALS

Catalog Number	Size		Maximum Movement Rating (in.)	Nominal Groove Width (in.)
	W	D		
Acme S-502	1 3/4	2	1/2	1 1/8
Brown B-1500	1 1/2	1 1/2	1/2	1
Brown B-1750	1 3/4	1 3/4	3/4	1 1/8
Acme S-500	2	2	3/4	1 1/4
Brown B-2000	2	2	7/8	1 1/4
Acme S-497	2 1/2	2 3/4	7/8	1 5/8
Brown B-2500	2 1/2	2 1/2	1	1 1/2
Acme B-496	3	3 3/8	1 1/8	1 7/8
*Acme B-610	3 1/2	3 1/2	1 1/8	2 1/4
Acme B-462	4	4 3/4	1 3/8	2 5/8
*Brown K-3000	3	2 1/2	1 3/8	1 3/4
*Brown H-4000	4	4 1/2	1 5/8	2 1/2
Acme B-613	5	5 1/4	1 1/2	3 3/8
*Brown K-5000	5	3 3/4	2	3 1/8
Acme B-614	6	5 3/4	2	4

Notes: Movement rating and nominal groove width subject to verification.

Nominal groove width to be corrected for temperature (see accompanying chart).

Designer will place the required thermal movement rating for each joint on the contract plans.

*Pressure generation may be less than 3 psi at 85 percent nominal width.

A Concept of Preengineered, Prefabricated, Prestressed Modular and Multimodular Sealing Systems for Modern Bridges and Structures

STEWART C. WATSON, Watson-Bowman Associates, Inc., Buffalo, New York

Single module, modular, and multimodular sealing systems appear to offer long-term, maintenance-free solutions to newly developing problems at bridge joints being brought about by new design sophistication. The need for armored joints and their damping effect together with improved embedment practices are discussed. Upward and downward vertical forces, rotation, deflection, and horizontal thrust movements and their effect on seal shapes are illustrated. The typical bridge-joint environment clearly dictates the need for heavy-duty seal configurations. Web, top, and side minimums, depth-to-width ratios, and pressure-generation requirements are presented and analyzed. Some methods of reliable deck temperature determinations and adjustment for temperature are given. Creep-shrink calculations and testing of modular and multimodular systems are illustrated.

•WHETHER A BRIDGE is of a suspension, cantilever, steel arch, continuous truss, cable stay, concrete arch, continuous plate, orthotropic, or box girder design, there are marked similarities in performance requirements for sealing systems. These similarities have indicated the feasibility of solving problems at the joints by a systems approach.

European engineers are frequently impressed with American mass production techniques and demonstrated ability to produce simple low-cost structures at a rapid rate; in like manner, a visitor to Europe cannot fail to marvel at the sweeping, continuous, architecturally pleasing freedom of design evidenced by our European counterparts. As a by-product of this increased latitude in design thinking, new problems have arisen that must be solved at the joints, as well as at the bearings, if we are to continue to progress. Single module, modular, and multimodular sealing systems based on the compression principle seem best suited to freeing engineers from the conventional because they offer the greatly increased performance levels so necessary to the new structural sophistication that is now spreading across North America.

This discussion is intended to better acquaint the bridge designer with what is being done today in modular compression sealing systems, their capabilities, some of the problems incurred, and certain fundamental construction practice considerations.

European modular systems, which have preceded the North American types, have field-proved their reliability on literally thousands of bridges, predominately of longer spans, with significant displacements and deformations. As an example of this widespread acceptance, there exist reference lists of well over 500 modular systems on bridges in Switzerland alone, installed over the past decade, utilizing the popular RUB System.

Paper sponsored by Committee on Sealants and Fillers for Joints and Cracks in Pavements and presented at the 49th Annual Meeting.

Rapidly increasing costs not only of new construction but of required maintenance on bridges and structures is dictating the need for improvements in jointing systems. Bridges have changed the economy of the world because they vitally affect the accessibility of land. It is therefore incumbent upon bridge designers to exercise every possible means at their disposal in the light of present knowledge to utilize maintenance-free concepts in the design of modern bridges and structures.

STRENGTH AT THE JOINTS A NECESSITY

Despite the heavy continuous loading to which the technically highly developed bridge superstructures are subject, economy and weight reduction are primary considerations in present-day designs. These considerations do not, however, apply to the design of deck expansion joints. Insofar as slender and light bridges are concerned, heavy, strong deck expansion joints appear to be much less subject to trouble and maintenance. Furthermore, if chosen from the outset, the total cost of a heavy-duty installation is lower than that of light design, which usually will require much maintenance, repair, and makeshift replacements, ultimately giving way to a heavy-duty installation after all.

IMPROVED EMBEDMENT PRACTICE FOR ARMOR-PLATING OF JOINTS

Some of the possible variables in concrete construction practice, unfortunately always present, have given rise to concern on the part of bridge design engineers throughout the world with regard to the ability of the average workman to produce good consolidation of concrete under the flat surfaces of embedded angle irons, channels, and other items that, as an integral part of a sealing system, can be pounded loose under repetitive traffic loading. Studies now exist showing the merit of very heavy steel cross sections to provide damping to truck-induced damaging vibrations. It is the design practice in a few countries to fasten armored joints to the main reinforcement of the structure in such a solid manner as to take no credit for lug embedment, treating them as a cantilever. Condition surveys of bridges in service in the United States as well as other countries suggest that this is an area for needed research.

VERTICAL FORCES

Experiences over the better part of a decade with monolithic bridge compression seals together with a massive dynamic compression seal failure on a large 3-mile-long bridge structure during the early part of 1968 have settled once and for all the question of the necessity for some mechanism to provide for resistance to vertical forces, both upward and downward.

Under a state of super-lubricity during heavy rains, it appears logical that a suction force is applied by rubber tires not unlike that from a rubber sink plunger.

Certain types of seal configurations that produce more stress at the top than at the bottom also tend to walk upward under rotational effects. Only field-proven seal configurations should be utilized because all shapes differ in their ability to resist upward vertical forces. Ideally, seal configurations used should incorporate a capacity to translate upward and downward vertical forces into a lateral force, with the forces being dissipated against the joint interfaces.

There can be no question but that downward vertical forces from traffic loadings and, to some degree, gravitational forces must be given consideration in the design of any sealing solution.

HEAVY-DUTY SEAL CONFIGURATIONS MANDATORY FOR BRIDGES

Certain experimental light-webbed seal configurations that have recently become available and are being suggested as adequate for bridge joint environments have given cause for concern on the part of design engineers. The first experimental bridge compression seals were actually hybrid devices consisting of thin-webbed contraction joint seal shapes that were bonded together to achieve the greater width and movement necessary to the needs of bridge joints. Even though the seals initially appeared to be successful insofar as longitudinal stroke of movement was concerned, it later became strikingly evident that there was an absolute need for heavier webs, heavier tops, and heavier sides to structurally resist not only vertical forces but also the very serious effect of foreign material being pounded by heavy traffic into the top and at the interfaces of the joints. In the long term, this intrusion tended to depress a light-webbed seal configuration into itself in a downward direction. The relatively thin-webbed cross sections, as compared to the field-proved standard North American heavy-duty bridge seals now in wide use, tended to take intrusions of foreign material at interfacial locations as the thin tops were unsymmetrically depressed under the effect of traffic loadings.

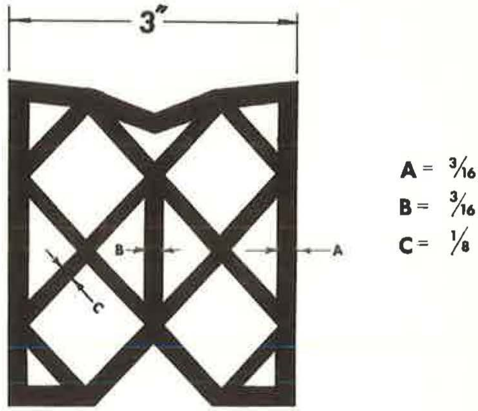

Figure 1. Field-proven heavy-duty bridge configuration.

Obviously, these typical service conditions are intensified during colder weather as the joints open to their extreme movement stroke. Frozen snow, ice, slush, maintenance grits, and other debris lying on top of the seal are slammed and ground into the configuration, a treatment that mandates the very ultimate in brute strength.

The design team responsible for field testing and development of these bridge compression seals after experience on literally thousands of bridges in every conceivable type of environment throughout North America made the considered judgment that web, side, and top thicknesses as shown in Figure 1 represent absolute minimums. Having been arrived at through the committee system, the recommendations take into account the dictates of bridge performance need, structural considerations, rubber manufacturers' capabilities, surface contact requirements, pressure-generation minimums, and ease of installation. To thin out webs in an attempt to obtain a greater movement stroke without taking into account the other needs is to contravene a proven concept.

MINIMUM PRESSURE GENERATION FOR BRIDGE SEALS

Specifications should be written to exclude flimsy, low-pressure configurations because they have been proved to have no place in the difficult bridge environment. The following ranges of pressure generation minimums appear to be adequate for bridge seals: For 1¼- to 2-in. seals, the minimum pressure at 85 percent compression (i.e., compressed 15 percent) should be 3 psi; for 2½- to 6 in. seals, the pressure should be 4 psi.

DEPTH-TO-WIDTH RATIO

A depth-to-width ratio has been established from long-term field experience and repeated condition surveys of seal performance. Proper seal depth is necessary to provide the desired area of interfacial surface contact and friction, and to maximize the ability to resist vertical migration. Most important, this depth ratio must be maintained to achieve leakproofing. Time-dependent post-installation interfacial spalling, edge attrition, dry shrinkage cracks, microcracking, interfacial cavitation, and other conditions necessitate that a maximum amount of surface contact area be provided. Specifications should require that the depth-to-width ratio for a bridge compression seal never be any less than 1 to 1.

The rapidly moving tendency toward a systems approach to sealing where more than one seal is used in a modular system requires that the foregoing pressure generation minimums and depth-to-width ratios be maintained in order to produce a force sufficient to move the separator plates without distortion through their stroke of movement.

ROTATION, DEFLECTION AND THRUST MOVEMENTS

Some typical movements occurring in bridge joint environments other than straight thermal opening and closing that must be absorbed by seal configurations are shown in Figure 2. Individual seals as well as modular and multimodular systems must have the ability to accept rotation of interfaces (Fig. 2a), resist alternating vertical deflection motion (Fig. 2b), and maintain their structural integrity under differential thrust displacements (Fig. 2c) without walking upwards, buckling of top portions, or other failure. Light-webbed seals with little pressure generation have not worked well in these types of movements, while the heavy-duty shapes (Fig. 1) have proved themselves thoroughly on thousands of bridges throughout the world. The basic seal design should be structurally adequate and exhibit its ability to maintain constant contact with the top edges of both joint walls during its full range of movement without misalignment or pulling away. There is an absolute necessity to field-test a seal configuration over a number of cycles of weather in a multiplicity of bridge-joint environments to prove its performance capability under the movement eccentricities noted.

SOME SOLUTIONS TO THE PROBLEM OF VERTICAL FORCES

Figure 3 shows the 1969 specification requirement of the bridge department in Utah for compression seals utilizing a seal cleat. A mating groove is machined into the joint armor as a solution not only to potential vertical migration but also as an effort toward leakproofing. One unique feature of the Utah system is maintaining the cleat location at the same position with respect to the riding surface so that as seal sizes and joint widths change from bridge to bridge, the cleat position remains vertically constant. It has been the experience in Utah that prestressed bridge decks in a number of cases have sustained time-dependent shortening, probably due to creep, and after long-term service this has necessitated replacing some joints with larger size seals.

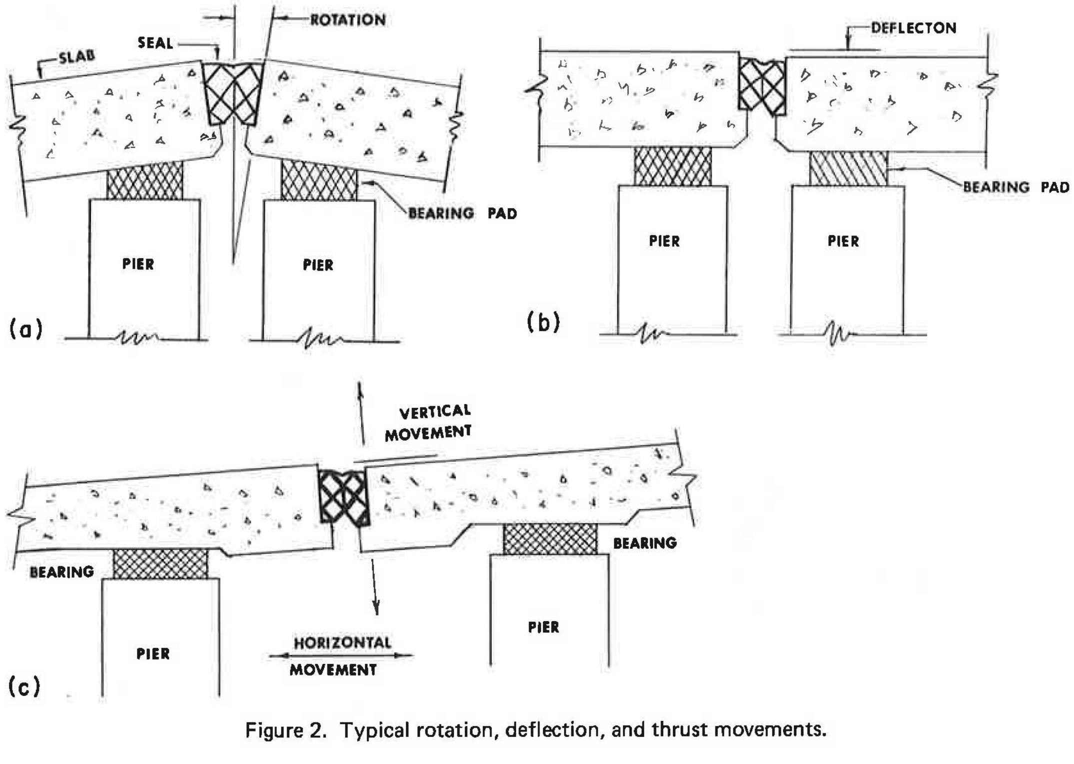

Figure 2. Typical rotation, deflection, and thrust movements.

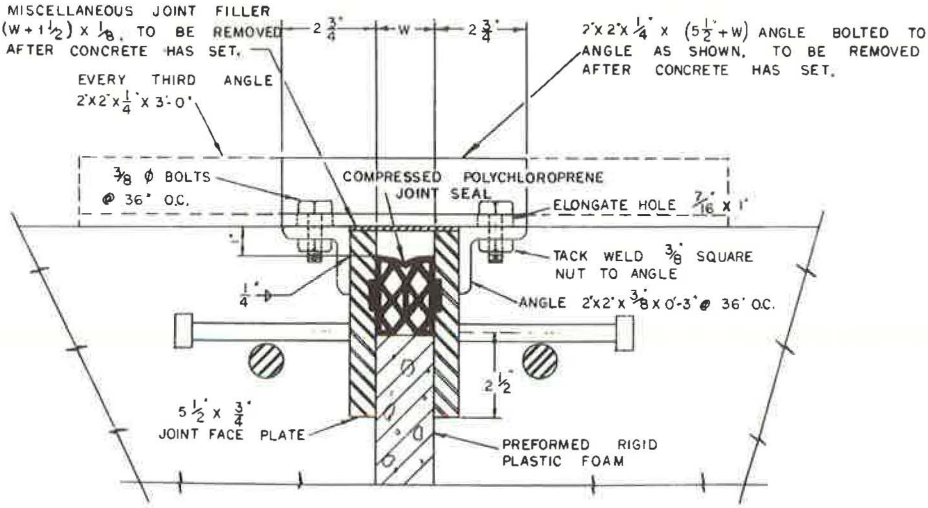

Figure 3. New State of Utah design incorporating seal cleats.

A Swiss-German solution now in wide use throughout central and south-central Europe consists of cantilevered plates, which, in addition to preventing upward vertical movement of a seal, also solves the problem of excessive joint width. The extent of the cantilever is limited to and must reflect the compression limits of the specific seal configuration being employed.

Methods have been developed to mechanically lock a configuration into place to preclude migration in either direction (Fig. 4). A secondary effect is to provide positive performance in long-term use of organic elastomers, the very finest of which during extended periods of in-service use would exhibit a gradual stress-related pressure decay. A third effect is to practically guarantee a 100 percent leakproof joint.

The importance of utilizing a good, high-solids, adhesive system with bridge-type compression seals must be underscored. It is now possible with the new types of adhesives that have been developed for compression seals to positively affix a seal to the joint interfaces with reliability. A recent example on Louisiana's Lake Pontchartrain comparison field tests of sealers clearly illustrates the importance of good adhesive systems for bridge compression seals. Because of the differential friction on bearings, movement unloading occurred, with the result that occasional joints moved in excess of the uncompressed width of some compression seals by as much as 1/4 in. (joints opened to 2 1/4 in. where a 2-in. wide seal was installed). Still, certain compression seals that had been installed with the new high-type lubricant-adhesives are performing effectively today because of being actually bonded in place. It should be the design goal of bridge specification writers to produce a rubber-tearing bond whose strength would be such that it would require a hammer and chisel for its removal. Because we cannot always predict with reliability the movements that will come on any type of structure, it is logical that we should use high-type lubricant-adhesive systems.

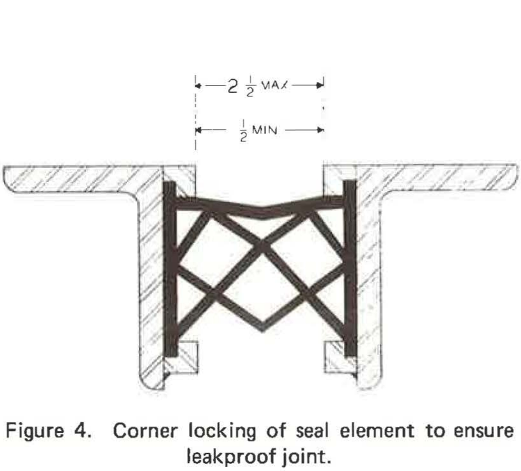

Figure 4. Corner locking of seal element to ensure leakproof joint.

DETERMINATION OF DECK TEMPERATURES

The success of any attempt to install a sealing system without giving attention to temperature considerations would be analagous to success at Russian roulette, being hit-or-miss at best. It is therefore a necessity, particularly on longer spans, to make a reliable judgment of the temperature of a given bridge deck or span in order to activate the sealing system, ideally, at the precise temperature of the span. This judgment can be rather complex, as evidenced by the work of Wah and Kirsey (1). In early spring or late fall, air temperatures and deck temperatures can differ 50 degrees because of temperature lag. Obviously, complex instrumentation could be implemented through which the temperature judgments might be made. However, Wah and Kirsey have indicated the pitfalls involved and the many variables that are possible.

It would be most desirable to be able to take the deck temperature without relying on complex and potentially unreliable as well as costly instrumentation. Because European bridge designers have been working for some time with longer spans, some actual working practices are included here. The British have used a measurement of shade air temperature beneath the deck at the time of setting a joint, or in the case of box girder construction, a measurement taken inside, to give an indication of the mean bridge temperature to within ±5 C (±9 F). This is normally accurate enough for setting an expansion gap capable of accommodating horizontal movements of up to 5 in., but when further accuracy is required, thermocouples or thermometers at representative points within the structure can reduce the error to ±2 C (±4 F). A German-Swiss engineer-contractor firm uses a small copper tube with its lower end squeezed or closed imbedded in the concrete at different locations for placing copper constantan thermocouples. A very simple system in use by one active British bridge expansion joint installation firm is to construct a small plaster of paris dam in a shaded area, fill it with water, and place a thermometer in the water.

Once a deck temperature judgment has been made, the sealing system is then prestressed to correspond. A sealing system that is not properly activated at the correct temperature or one that does not include this consideration in its design is capable of self-destruction, damage to the bridge, or a combination of the two.

PLACEMENT OF SYSTEM IN A DECK

Two methods of placement exist—the blockout method and cast-in-place method.

Blockout Method

The blockout method appears to be the safest method and is probably preferred because it is simple and eliminates many problem areas in construction. There is the consideration of having a construction joint at the blockout, but recent improvements in placement methods have operated to solve the ridability, concrete-to-concrete bonding, and leakage difficulties.

Figure 5 shows a modular system with a 6-in. movement capability that has been adjusted for temperature-width and is to be welded to the main reinforcement of the bridge deck. This offers ideal performance since the system becomes a part of the bridge structurally. The excellent ridability obtained by this method is very simply and positively achieved by means of low-cost, recoverable, positioning support members that longitudinally span the blockout and accurately suspend the sealing system while the concrete is being placed.

Cast-in-Place Method

The cast-in-place method has the advantage of eliminating the construction joints that are a part of blocking out. However, it presents opportunities for aborting the system if field personnel are unfamiliar with the intricacies of these somewhat sophisticated sealing devices.

Once a sealing system has been prestressed for temperature width and fixed for placement, the threaded rods, centroid to the device, must be removed in order for it to reflect the anticipated movement eccentricities of the structure. It then becomes obvious that the setting of the proper prestressing, the fixing of the device, and the

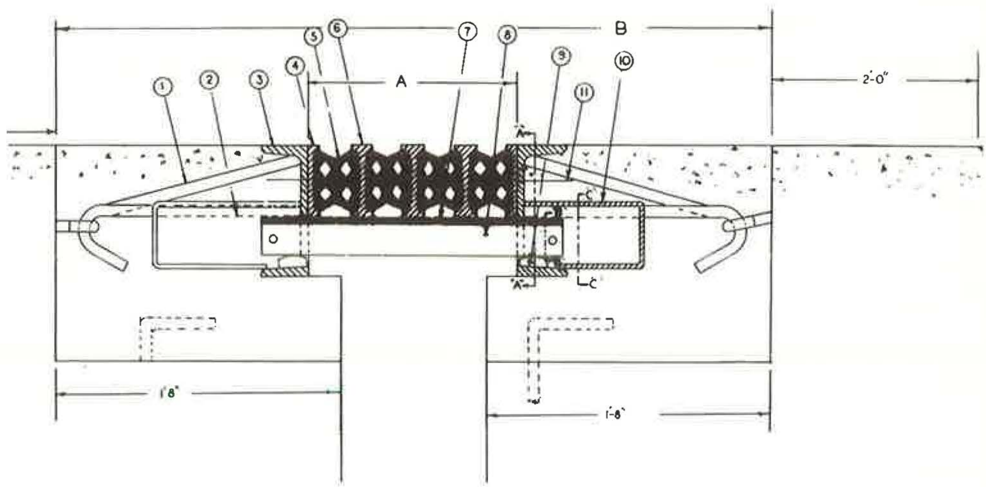

Figure 5. Plan view of 4-tube, 6-in. movement modular system.

placement of the concrete must necessarily be in concert with elevating thermocentripetal displacement. In plain language, prestressed sealing systems should be installed and placed beginning in the morning or as the deck temperature is rising, because the prestressing mechanisms will then self-loosen themselves for ease of removal. An improvement to single module and modular systems has now been developed that would permit some slab-end regression, such as would be occasioned by a dynamic temperature drop (cold front moving in), excessive wind-chill effects, or inordinate creep or shrink, prior to relieving or activation of the prestressment mechanism.

IMPROVED BEARINGS AND THEIR RELATIONSHIP TO THE SEALING PROBLEM

With respect to the new continuous bridges of longer spans, the old rigid plus the newer elastomeric bearings have definite structural, rotational, unilateral, multilateral, and longitudinal movement as well as height and economic limitations. Particularly with longer spans where higher loads and significant displacements occur, the design of the bearings and the design of the sealing system should be interrelated.

The new pot bearings were developed in Germany and are designed to minimize strains on a structure as well as its foundations, and eliminate undesirable friction and costly maintenance by utilizing primarily inorganic materials such as stainless steel. A typical pot bearing consists of a rubber disk set inside of a shallow piston/cylinder assembly. Behavior resembles a hydraulic cylinder containing a viscous fluid. Because rubber is in reality a liquid, the neoprene used in this application cannot be affected by prolonged stress inasmuch as it is actually taking the place of the oil in a piston assembly. This allows the bearing to accept rotation with negligible shift in the center of pressure. For unilateral and multilateral bearings, one face is equipped with a Teflon pad sliding against a polished stainless steel plate permitting horizontal movement. The resulting sliding friction coefficient of 1 percent or less permits lowest bending moments and shear forces.

Bridge engineers should proceed with caution in the selection of bearings incorporating fluorocarbon sliding surfaces. Certain new designs have recently appeared in which resultant horizontal forces and frictions could produce a crushing overstress to the fluorocarbon. Furthermore, the concrete stress in the outer area of the bearings can become dangerously high. One should be suspicious of the calote-type bearings where horizontal forces are absorbed in the outer sector of the bearing only and not centered.

SPECIAL DESIGN FOR SNOW-ICE ENVIRONMENT

Because jointing systems on bridges in snow and ice areas are more vulnerable to the increased demands of this environment, special attention in design is obviously a necessity. It would appear logical that all exposed surfaces of the jointing system should be lower than the riding surface of the deck by ¼ in. In addition, all exposed corners or edges should exhibit a radius. Special attention should be given to salt brine attack, and vulnerable portions of the system should be designed for ease of replacement should unusual damage from plows occur. In very low temperature areas where temperatures are considerably below -20 F for sustained periods of time, the seal-lock concept should be given consideration.

MODULAR SYSTEMS AND DAMPING EFFECT

Interest in orthotropic bridges has increased greatly in North America, and a number of structures utilizing this design currently under construction are incorporating modular and multimodular sealing systems not only for their ability to perform with adequacy under large longitudinal displacements but also because of their natural damping effect in compression. Obviously welcome economies are incumbent through orthotropic designs, but an inherent loss of stiffness and responses of these decks to the forces of excitation offer a challenge to the designer of the jointing system.

The Halifax-Dartmouth Narrows Bridge has specified a multimodular system for the expansion joints under the main towers with a performance requirement of 18 in. in longitudinal movement and its resultant damping effect is expected to contribute toward a reduction in vibration on this orthotropic structure (Fig. 6).

Papineau Bridge, which will link the north end of Montreal Island with the mainland over Riviere des Prairies, has been designed with cable stays utilizing two slender 126-ft high towers and an orthotropic deck, the center span being 336 ft. Aerodynamic model studies that have been conducted on this design have shown that vibrations of 2-in. amplitude could over a period of time be destructuve while an amplitude of 9-in. during an extremely high wind could also be serious. Four modular systems with 9 in. of movement each have been designed for installation in this interesting Canadian structure. The Bayonne River Bridge in Quebec, the first orthotropic bridge in North America to utilize a concrete wearing surface on its decks, has incorporated 4 modular packages, each with 4½ in. of movement.

CREEP AND SHRINKAGE

Construction practice permitting, a modular system should be installed when most of the creep and shrink has taken place. When bridges are constructed of prestressed

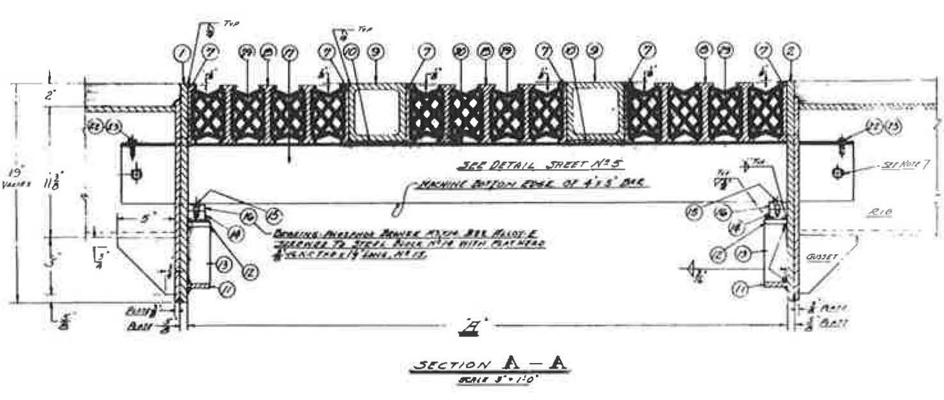

Figure 6. Modular sealing system incorporating 18-in. movement capability for Halifax Narrows Bridge.

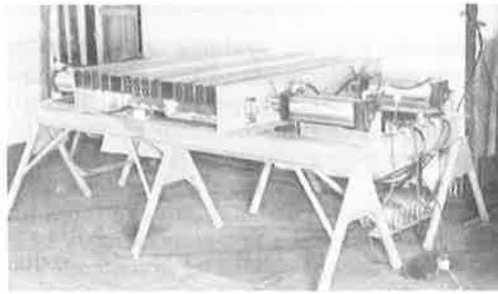

Figure 7. Full-scale testing of an 18-in. modular system.

beams and in situ concrete, the rate and amount of creep appear to be difficult to calculate. To a greater or lesser extent, under continuous loading all construction materials and all types of structures will incur irreversible dimensional loss. Shrinkage of a concrete structure, not to be confused with the lessening of a dimension due to creep, occurs mainly due to the moisture loss during curing.

A typical example of movement calculation using Swiss SIA Standards follows. Assume the effective length of bridge = 100 m (332 ft); the centric stress from prestressing = 60 kg/cu m; the temperature at fitting of joint is approximately +10 C; and a modular system of 100 mm (4 in.) movement. The calculation for pre-adjustment of the Modular System is as follows:

Creep	1.5 cm	
Shrink	2.0 cm	
Temp. decrease down - 10 C	2.5 cm	
Total shortening	6.0 cm	(theoretical pre-adjustment measurement)
Temperature increase up to +30 C	1.5 cm	
Effective displacement of joint	7.5 cm	
Reserve movement of modular system	2.5 cm	
Total movement of modular system	10.0 cm	

It may be safe to say that the phenomenon of creep and shrink is still not thoroughly understood or completely defined. In view of this, it is of utmost importance to develop a practical, reliable, empirical method of creep-shrink calculation that works well for the modular system employed, construction method used, type, age, and geometrics of materials, loads involved, and environmental conditions, because the effects are irreversible and must be pre-adjusted at the moment of activation of any sealing system.

TESTING OF MODULAR SYSTEMS

Even though single module, modular, and multimodular systems can be fabricated for a wide range of movements and performance conditions, the assumption cannot be made that if a single module performs well, a four-, eight-, or twelve-tube modular system merely involves sandwiching up whatever elements are necessary to match calculated movements. Full-scale working sections should be run through their total anticipated ranges and types of movement in advance of fabrication to predict the reliability and practicality of a design. Figure 7 shows a full-scale working device 6 ft long built to accommodate 18 in. of longitudinal movement.

REFERENCE

1. Wah, T., and Kirsey, R. Thermal Characteristics of Highway Bridges. Southwestern Research Institute, 1968.

Hot-Poured Sealants

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Hot-poured sealants are either straight asphalt cements or asphalts that have been modified with fillers or rubber or both. They are the lowest in cost and the most widely used sealing materials available today. Specifications for hot-poured sealants are being written based on laboratory tests that often do not correlate with actual field performance. Sealant performance depends on type, quality, and quantity of materials used. It is suggested that materials requirements be included in sealant specifications. Various types of rubber used and manufacturing costs involved in the production of rubberized asphalt sealants are discussed. Installation procedures and problems affecting service life of the sealant are given. Cost comparisons are made between hot-poured sealants and other joint-sealing materials. Research is suggested that should result in a better understanding and improved quality of hot-poured sealants.

• WE ALL KNOW WHY contraction joints are built into portland cement concrete road surfaces and we have witnessed the catastrophic failure that can occur when they do not function properly. The engineering principles involved in design of contraction joints are well known and widely recognized. It should therefore follow that the joints must remain functional if ultimate service is to be obtained from the road surface. The joint must, however, be sealed to prevent water from leaking through and destroying the subbase or to prevent incompressible materials from entering the joint and rendering it useless.

Certainly the service that a joint sealer is expected to withstand is severe, and there has been a continuing search for materials that will perform this function at a reasonable cost. Through the years many materials have been tried. However, today we find only four major classes of joint sealers being used. These are (a) preformed compression seals, which can only be used in new construction or placed in joints that conform to a given shape; (b) elastomers, such as polyurethanes, polysulfides, and others; (c) cold-poured sealants, or asphalt cutbacks; and (d) hot-poured sealants. It is the purpose of this paper to discuss hot-poured sealants.

Hot-poured sealants are the most widely used materials for sealing joints and cracks. The "hot-pours" are asphalt cements that can be modified with mineral fillers or rubber or both. Asphalt cutbacks and emulsions are not considered hot-pours because most of them are used at ambient temperatures or heated to only 120 to 140 F.

Coal tar has also been mentioned as a hot-poured sealant, although no state in the Northeast uses it. In discussions with highway personnel, the word "tar" is often used (tar kettles, tar pots) even though asphalt is the material referred to in most cases.

SPECIFICATIONS

Many states use a straight asphalt cement (85-100, 50-60) to which they assign a state specification number. Others modify the asphalt with mineral fillers or rubber.

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The mineral filler is usually finely ground talc or limestone, 65 percent of which will pass a No. 200 sieve. These materials are used to harden the asphalt and give it "body".

Because some of these sealants have been used for many years, it is not surprising to find that many highway departments do not know the origin of their own specifications. There is, however, almost universal agreement that these sealants are not performing the function for which they are used.

The addition of rubber to asphalt improves its flexibility, ductility, adhesion, and cohesion properties. It has therefore been used through the years in the production of sealants designed to meet the federal specifications for hot-poured sealing compound. (SS-S-164, issued in February 1952, is an example of a specification for hot-poured sealants.)

The first rubberized asphalt sealants were made with a high percentage of rubber (20 to 30 percent by weight). They were high-quality sealants that performed very well. As the use of rubberized asphalt became more widespread, more and more companies entered the field. With increased competition, prices were forced down, and, as a result, quality and performance suffered. Performance failure cannot, however, be fully blamed on quality alone, because other factors such as joint width, joint spacing, and installation practices have an important bearing on joint performance.

The federal specification (SS-S-164) called for physical testing of a sample in the laboratory to meet certain test requirements such as safe heating temperature, penetration, cold bond, and flow. This is a performance specification and there is no mention of amount, type, or form of asphalt, rubber, or other material to be used. There is no objection to a performance specification, but state highway engineers are in agreement that the performance requirements have not been properly spelled out. Should there not also be a material requirement stated in the specification?

Originally 20 to 30 percent (by weight) of rubber was used to prepare sealants designed to meet SS-S-164. As the years went by, smaller amounts, different types, and various forms of rubber were used to lower costs. As the rubber content was reduced, larger amounts of filler and other materials were added. Even though fillers can be used advantageously, excessive use can contribute to the poor quality and short life of the finished product.

In 1967, a new federal specification (SS-S-1401) was issued. This one reads:

> The sealing compound shall be composed of a mixture of materials compatible with asphalt with or without rubber and which will form a resilient and adhesive compound, will effectively seal joints and cracks in pavement against the infiltration of moisture throughout repeated cycles of expansion and contraction, and will not flow from the joint or be picked up by vehicle tires at an ambient temperature of 125° F. (52° C.). The sealing compound shall have a uniform pouring consistency suitable for completely filling the joints without inclusion of large air holes or discontinuities. The pouring temperature shall not exceed 450° F. (232° C.).

Again we find that this specification does not spell out the type of materials to be used but has simply changed the physical tests and added two new ones, resiliency and compatibility. The major objection to performance specifications (and this is shared by highway maintenance people) is that the sealants will often pass the laboratory tests but will fail in field performance. Many engineers feel that uniform field performance can only be obtained by specifying and controlling the materials to be used in production of the sealant.

New York State Addenda No. 14-M34A specifies the percentage and type of rubber to be used, along with laboratory tests and field installation procedures. Maintenance crews on several toll roads use an asphalt and add the rubber directly into the kettle on site. This approach has the obvious advantage that the percentage and type of rubber are known and can be controlled. Many states are currently testing this method.

USE OF RUBBER IN THE SEALANT

Types of rubber available for the manufacture of joint sealants fall into three major classifications. These are natural, synthetic, and reclaimed. Natural rubber comes from trees, synthetic rubber is manufactured, and reclaimed rubber can be produced from products containing either natural or synthetic rubber, but is most often made from products containing a mixture of both. The various types of synthetic rubber include SBR (styrene-butadiene), butyl, nitrile, neoprene, polybutadiene, polyisoprene, and ethylene-propylene. SBR accounts for 70 percent of all the synthetic rubber used in the United States (twice as much as natural rubber).

Rubber to be used in an end product is normally vulcanized by the addition of sulfur and usually contains other ingredients, such as reinforcing agents, accelerators, intermediates, and plasticizers. Reclaimed rubber is produced by replasticizing the vulcanized rubber by means of heat, pressure, and chemical agents (devulcanization).

Rubber comes in three physical forms, slabs or bales, granular, and liquid or latex. An important consideration in the production of any quality product is securing and maintaining uniform raw materials. When selecting the rubber to be used in the preparation of rubberized asphalt joint sealers, lower cost and greater uniformity can usually be obtained by choosing ground vulcanized rubber or granular reclaimed rubber that contains a high percentage of SBR.

The best raw material source for either of these materials is used passenger tires. Ground vulcanized rubber or reclaimed rubber produced from used passenger tires that have been properly handled (sorted, with metal and fabric removed) will be uniform and contain few impurities. Specifications can be, and are, written around these types of rubber.

A common source of scrap rubber available in any city is buffings from truck, bus, and passenger tires that are buffed from the tire prior to recapping. (Truck and bus tires have a high percentage of natural rubber.) This material will vary considerably in composition and particle size and will contain various amounts of magnetic and nonmagnetic contamination (filings, various metals, glass, stones, and organic materials). Even though virgin synthetic rubbers can be used, costs are higher and much greater reheating control is required in the field to prevent degradation.

The cost of rubber in a sealant is not always a major factor in the sealant price. Ground vulcanized or granular reclaimed (devulcanized) rubber sells for about 8 to 12 cents per pound, and the addition of 25 percent by weight to asphalt increases the raw material cost of the sealant by only 2 to 3 cents per pound. The cost of a manufactured rubberized sealant is considerably higher than that of a straight asphalt sealant because of the extra labor costs, increased power consumption, and more capital equipment necessary to heat, mix, and stir the sealant during production.

The use of small quantities of vulcanized or reclaimed rubber does not appreciably reduce the raw material cost, but may affect the manufacturing cost of the sealer. Liquid latex is also used to produce rubberized sealers and its use will normally result in a lower manufacturing cost. However, latex is quite expensive, and 25 percent ground vulcanized or reclaimed rubber can be added at the same material cost as 4 percent latex.

A low-cost rubberized sealant can be produced on the job site by mixing granular reclaimed rubber directly into the asphalt kettle. In this manner, the type, amount, and gradation of rubber can be controlled, and manufacturing variables can be eliminated.

INSTALLATION PRACTICES

The recommended procedure for using a hot-poured sealant is to heat below 450 F and pour. In actual field use, most kettles are the flue type, heated directly at the bottom by kerosene burners that can subject the sealants to localized temperatures of 800 F or higher. The kettles are equipped with metal covers, and in many cases fire extinguishers and canvas are available to put out sealant fires. The hotter the sealant, the easier it is to pour and prevent solidification in the pouring-pot nozzles. While the sealant is being used, makeup material must be added to ensure enough for a full day's work. The addition of cold sealer reduces the temperature of the mass in the kettle. Therefore, the hotter the sealant, the more chance of maintaining production, i.e., the number of joints and cracks that can be filled in a given period of time. On cold days, a hotter sealant penetrates deeper into the joints with better adhesion to the sides.

The difference between actual field application practices and specified installation procedures is unbelievable. Hot-poured sealants have been poured into joints that were damp and water-filled and that had compression seals, wood spacers, and old sealants in them. These conditions are certainly not recommended by either sealant producers or highway engineers; nevertheless, they do exist in the field.

An educational program should be set up between the joint sealing crews (maintenance) and administrative personnel. No material should be expected to give satisfactory performance unless installation crews are properly equipped and trained for making the installation.

The hot-pours are the lowest-priced sealants available in the United States. Installation costs for a hot-pour run from 20 to 30 cents per linear foot, whereas elastomers are between 40 and 60 cents and preformed compression seals are between $1.00 and $3.00. Therefore, on an installed-cost basis, the elastomers are 2 times higher, and the preforms are 5 to 10 times or an average of about 8 times higher than the hot-pours. If you assume a 5-year service life for a properly prepared and installed hot-poured sealant, you would have to realize 10 years' performance from an elastomer and 40 years from a preform to be on a comparable cost basis. In other words, to break even for each year of service life of a hot-pour, you must obtain over 2 years of service from an elastomer and over 8 years of service from a preformed compression seal.

ADVANTAGES OF HOT-POURED SEALANTS

Hot-poured sealants offer many advantages over other sealant materials:

1. Ease of application—Just heat and pour.
2. Low cost—The lowest cost material available.
3. Deeper penetration—The hot sealant with its lower viscosity can go deeper into the joints and cracks to seal areas where water can penetrate.
4. Ready conformation to joint shape—The use of the shape factor is only valid for highly elastic, high-recovery materials. The hot-poured sealant is a low-recovery material and therefore a perfect shape factor is not required and one may well be better off without it.
5. Sealing of spalled or ragged joints—Hot-poured sealants are the only material that can be used in this type of work. Elastomers and preforms are useless in this application.
6. Requiring no special equipment—Equipment is familiar to the maintenance and installation crews.
7. Easier application by unskilled labor—No special training of crews or elaborate instructions are required.

Full utilization of these advantages is not realized if the joints and cracks are not properly cleaned and prepared before sealing.

To summarize, the hot-poured rubber-asphalt sealant is the highway engineer's best buy on a price-performance basis. If a pavement must be resealed with rubber-asphalt sealant 5 times in its expected life (25 years), the savings are still great compared to other materials. We could pour and seal over 8 times (or for 40 years) for the equivalent cost of one compression seal installation.

In order to obtain the optimum performance from a hot-poured rubber-asphalt sealant we must, however, have

1. Quality control of the rubber (including type and gradation),
2. Sufficient amount of rubber, and
3. Proper installation.

All three of these conditions can be specified and checked by any highway department. This is one of the simplest ways of improving hot-poured rubberized sealants.

REQUIRED RESEARCH

There are basic questions that remain to be answered about hot-poured sealants. Research in these areas could vastly improve our knowledge.

1. What asphalt should be used? Has anyone ever field-tested various asphalt cements to determine which ones give best resilience, adhesion, and cohesion?

2. What quality and quantity of rubber should be used with what asphalt for optimum flexibility, adhesion, cohesion, and resilience? Discussions here are based on laboratory and technical service work by our company with various joint seal manufacturers and highway departments. Details of time and temperature vary considerably with the type of mixer used and also the asphalt source. The rubber is usually added to the asphalt, which is held at about 430 F. It should not be heated over 460 F. After all the rubber is added, the mixing cycle will vary with the temperature and the type of equipment used. The most common type of mixer used is a jacketed paddle type varying in capacity from 200 to 3,000 gallons. It would be difficult to standardize the production of hot-poured joint sealants because it is considered an art that most manufacturers keep secret.

3. What test should be used to determine quantity of rubber in asphalt? There are several chemical tests to determine the percentage of rubber in asphalt, but most of these are not accurate because duplication of results is difficult, even within the same batch. Work should be done with instrumentation—perhaps a mass spectrometer.

4. Is rubber dissolved or dispersed in asphalt? There has been considerable discussion as to whether rubber really dissolves or simply disperses.

5. Does a rubberized hot-poured sealant have to be smooth? Some states insist on smooth joint sealers. However, as long as graininess of the product does not interfere with other properties of the sealant, such as pour, it can be desirable. In fact, the grain does act as a rubber reserve for prolonged and high heating. Some of the better sealants have been grainy.

6. What about low-recovery-type sealants? A study should be made to determine definite values for the adhesive and cohesive strength as well as definite values of stress relaxation and recovery. The effects of shape on these physical properties should also be studied.

CONCLUSIONS

Hot-poured sealants are the most widely used materials but the ones about which the least appears to be known. The hot-poured sealants are used on first-class highways as well as on county, city, and village roads. They will be with us for many years to come because of the advantages pointed out here. A program of research and development would enhance their utility even more.

Evaluation of Coatings on Coastal Steel Bridges, 16-Year Period

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The California Division of Highways investigated a wide spectrum of coatings for the protection of steel from corrosion in an aggressive marine environment for 16 years in the period 1952-1968. Evaluation of test sections of two bridges to which the coatings were applied showed that the best system after a 10-year exposure was an inorganic post-cure zinc-pigmented sodium silicate primer having a vinyl finish coat. This system was rated 9+, with 10 being perfect. Evaluation of the inorganic self-cure zinc-pigmented silicate primers will not be available for another 6 to 10 years. All other systems provided definitely inferior protection, the next best being an all-vinyl type.

•THE SEARCH for a satisfactory paint system for the protection of exposed structural steel members of bridges in a corrosive marine environment has been conducted intensively by the Bridge Department and the Materials and Research Department of the California Division of Highways since 1952. Properly formulated and applied conventional state and federal specification primers and proprietary primers containing various drying oils, alkyd resins, and phenolic resins in the nonvolatile vehicle and the well-known red lead, basic lead silicochromate, and zinc chromate rust-inhibitive pigments will normally protect steel from corrosion in inland areas (nonmarine) for 25 to 30 years if used in conjunction with suitable finish coats. These coating systems have been found to have a maximum life of 4 to 6 years when used on bridges located in a severely corrosive marine environment as described in this paper.

A 10-year study of coatings applied in 1958 shows that a post-cure inorganic zinc silicate primer with vinyl finish coats has provided almost perfect corrosion protection and that no organic vehicle system tested, with or without rust-inhibitive pigments, will provide equal protection. Studies are continuing in the evaluation of the self-cure inorganic zinc silicate primers, results of which will not be available for 6 to 10 years.

In some cases the photographs do not indicate the severity of the corrosion reported in the text of this report, especially where the corrosion was pronounced on the bottoms of the lower flanges and on the braces, which are not shown in all of the pictures.

TEST SITES AND PAINTING HISTORY

Although a few other bridges were involved, this report is confined to coatings applied to the bridges spanning the Leffingwell and San Simeon Creeks on a section of California Route 1 in the vicinity of the town of San Simeon. Both bridges are close to the shoreline of the ocean, presenting an ideal location where protective coatings could be applied and tested in a very corrosive salt-air environment.

These steel bridges were constructed in 1932 when the Coast Route was first opened to traffic. The paint systems applied prior to the start of this research project in 1952

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had an average life of 3 to 4 years on these bridges, whereas the same paint systems applied on inland structures removed from salt-air corrosion had a life of 25 to 30 years.

The following listing gives the year each of these bridges was repainted subsequent to 1932. The year given does not necessarily mean that repainting should not have been done earlier: San Simeon Creek—1932, 1936, 1938, 1943, 1946, 1949, 1953, 1959, and 1964; and Leffingwell Creek—1932, 1936, 1938, 1943, 1946, 1949, 1952, and 1958.

Prior to 1953, contracts for repainting specified the number of coats of primer and finish coat, but no thickness of coatings. At the start of the paint research project in 1952, state forces from the Division of Bay Toll Crossings did the painting of Leffingwell Bridge. Although no advance requirement for film thickness was required, each coat of paint was heavily applied to the degree of no sag, and subsequent dry film thickness readings were taken by the resident engineer of the Bridge Department. Beginning in 1953, all state highway painting contracts specified the dry mil thickness of primers and finish coats.

LEFFINGWELL CREEK BRIDGE, 1952 REPAINTING

The Leffingwell Creek Bridge was the first of the two bridges in the coastal area near San Simeon that was repainted under the research project of the Materials and Research Department and the Bridge Department. Between 1932 and 1952 it had been painted six times. It is a low-level structure containing eight steel stringer spans with a concrete deck supported on concrete piers and abutments. The 24-ft wide concrete deck of this bridge as well as that on San Simeon Creek shields the steel girders to a limited extent from rain, thereby allowing salt to accumulate in heavier deposits than would occur if the steel were exposed more as in a superstructure. Prior to painting, the steel on both bridges was sandblasted to the appearance of cast aluminum that is essentially equal to a Steel Structures Painting Council "white metal blast."

In the 1952 repainting research project, the stringers of the Leffingwell Bridge were divided into 48 strips, each of which was used as a separate test panel for the application of the experimental coatings. The length of the panels varied between 4 and 7 ft and the area varied between 150 and 240 sq ft.

Seven different primer paints and two different finish coats were used. The primers used were as follows:

System 1. A semiquick-drying red-lead primer—The nonvolatile vehicle was approximately 1:1 by weight of raw linseed oil and alkyd solids conforming to Federal Specification TT-R-266, Type III.

System 2. A zinc chromate primer—The nonvolatile vehicle was an alkyd similar to the present Specification 681-80-430.

System 3. Red lead and oil—The nonvolatile vehicle was raw linseed oil. This paint was used as a primer on state bridges prior to 1953.

System 4. A red lead-phenolic varnish primer.

System 5. A red lead-linseed oil-phenolic varnish based on the San Francisco-Oakland Bay Bridge Formula X-6.

System 6. Red lead-vinyl resin vehicle primer.

System 7. Red lead-epoxy ester-vinyl resin vehicle primer.

The finish coats on the 1952 painting of Leffingwell Creek Bridge were as follows:

1. Vinyl aluminum, a vinyl resin vehicle; and
2. No. 5 aluminum finish coat from the 1949 Standard Specifications (the vehicle for this finish coat was a phenolic-china wood oil varnish).

The vinyl aluminum was applied over the vinyl-red lead and vinyl-epoxy-red lead primers. The No. 5 aluminum finish coat was applied over all other paint systems. Selected test panels were first coated with vinyl wash primer (Federal Specification MIL-P-15328) and compared with other panels of the same paint system without the vinyl wash primer.

Critical examination of this bridge in 1958 prior to repainting shows that certain of these paint systems were better than others, but corrosion was severe enough after 6 years in all systems to require repainting. Figure 1 shows conditions typical of all seven systems.

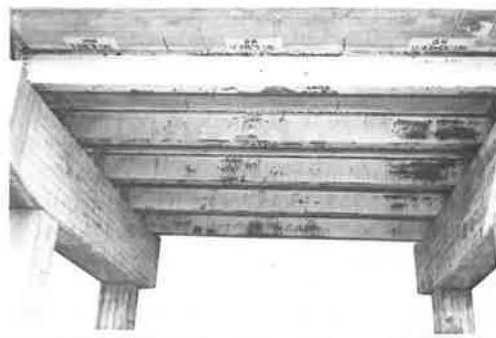

Figure 1. Panel 8N—Zinc chromate primer and No. 5 aluminum finish coat; panel 9N—red lead and oil primer and No. 5 aluminum finish coat; panel 10N—red lead and oil and aluminum finish coat.

SAN SIMEON CREEK BRIDGE, 1953 REPAINTING

The San Simeon Creek Bridge was painted in 1953 for the seventh time in 21 years for an average paint coat life of 3½ years and a maximum time between paintings of 5 years. Its record is one of rapid and consistent failure of all varieties of paints used. Rusting and consequent paint failures have generally occurred at the same locations after every painting as was the case of Leffingwell Creek Bridge. At the time of repainting of the San Simeon Creek Bridge in 1953, the previous asphaltic mastic coating was a complete failure. In the corrosion process, up to ⅛-in. deep pits were formed on the bottom flange.

In the 1953 repainting, the surface was sandblasted to the appearance of cast aluminum as described in the Standard Specifications. All sandblasted surfaces, except those specifically omitted, received a first coat of vinyl wash primer, Federal Specification MIL-P-15328. The paint systems used on the test sections were as follows:

System 8. About 4 mils of a highly recommended brand name alkyd red-lead primer was applied. No vinyl wash primer was applied at the request of the commercial supplier of the coating system. An aluminum finish coat supplied by the manufacturer was applied about 2 mils thick over the prime coat.

System 9. The so-called Harvey System, whereby a barrier coat is applied over the first coat to prevent subsequent coats from lifting the first coat, was used. Sections of the bridge were coated with two distinct Harvey Systems, each containing a different inhibitive primer. Total dry coating thickness was approximately 4½ mils in each system. Harvey System I consisted of (a) one coat of an alkyd-linseed oil-red lead primer, State Specification 52-G-53, with no vinyl wash primer being used; (b) a barrier coating of white traffic-line paint, California Division of Highways Type IV, containing a china wood oil oleoresinous varnish vehicle, chlorinated rubber, and a highly aromatic solvent; and (c) vinyl aluminum finish coat, State Specification T53-G-49. Harvey System II was the same as Harvey System I except that the primer was a zinc chromate type, State Specification 52-G-51.

System 10. Metalizing system—No vinyl wash primer was used on the sandblasted steel. Aluminum and zinc metal coating was sprayed in molten state onto sandblasted steel. Application was approximately 5 mils. Half of each of the aluminum- and zinc-coated test panels was coated with 3 mils of a vinyl aluminum coating applied over a vinyl wash primer.

System 11. Linseed oil-alkyd-red lead primer, State Specification 52-G-53, was applied. One section was coated with 3 mils of the primer and 2 mils of No. 5 aluminum finish coat containing a china wood oil-phenolic varnish. Another section had 4.5 mils of the primer and 3.5 mils of the finish coat.

System 12. A two-component epoxy-red lead primer was applied 4 mils thick followed by 2 mils of aluminum vinyl finish coat.

System 13. An all-vinyl paint system was applied, with one section 5 mils thick and the other section 8 mils thick. The vinyl primers were State Specifications T53-G-40 and T53-G-41 applied in alternating coats. The T53-G-49 vinyl aluminum finish was applied in one coat.

Inspection of the San Simeon Creek Bridge in 1959 prior to repainting showed varying degrees of breakdown of the experimental coatings applied in 1953 with resultant corrosion of the steel. The all-vinyl system (System 13) appeared to give the best protection, having a rating of about 7 on a scale where 10 would indicate no corrosion. Most severe corrosion was shown by Systems 8, 10 (without vinyl top coat), and 12, each being rated about 3. Systems 9 and 11 were rated as 5.

The aluminum and zinc metalizing coatings system (System 10) was very porous and there was much underfilm corrosion. However, where vinyl finish coats were applied to the metalizing, results were very satisfactory. Better protection of steel and easier application can be achieved by the inorganic zinc silicate coatings with vinyl finish coats as described in the 1958 Leffingwell Creek Bridge painting.

LEFFINGWELL CREEK BRIDGE, 1958 REPAINTING

The following coatings were applied to the Leffingwell Creek Bridge in the 1958 repainting (vinyl wash primer was applied to the sandblasted steel unless otherwise noted):

System 14. No vinyl wash primer was applied to the sandblasted steel; 3 mils of inorganic zinc-pigmented sodium silicate primer was applied and subsequently cured with a spray application of a phosphoric acid solution curing agent. The reaction products of the curing agent and zinc-silicate primer were scrubbed off the coated steel with water and stiff brushes. Vinyl wash primer, Federal Specification MIL-P-15328A, was applied followed by 2 mils of vinyls, State Specifications T58-G-40 and T58-G-41, in alternating coats and a final coat of 1 mil of State Specification T58-G-49, vinyl paint, aluminum finish coat. Total film thickness was 6 mils.

System 15. The following were applied: 3 mils of semiquick-drying red lead primer, State Specification 58-G-53; 2 mils of white traffic paint, State Specification 55-G-95; 2 mils of vinyl paints, State Specifications T58-G-40 and T58-G-41, in alternating coats; and 1 mil of State Specification T58-G-49, vinyl paint, aluminum finish coat. Total film thickness was 8 mils.

System 16. Epoxy paint, 100 percent solids, made with an epoxy resin of viscosity 40-100 poise at 25 C and an epoxide equivalent of 180-195, 20 percent TiO_2, and 5 percent Cr_2O_3, cured with an epoxy amine adduct Shell Epon Curing Agent U, was applied by hot spray 15 to 20 mils thick.

System 17. The following were applied: 4 mils of vinyl paints, State Specifications T58-G-40 and T58-G-41, in alternating coats; and 2 mils of vinyl paint, aluminum finish coat, State Specification T58-G-49. Total film thickness was 6 mils.

System 18. The following were applied: 4 mils of semiquick-drying red lead primer, State Specification 58-G-53; and 2 mils of phenolic iridescent green, State Specification 58-G-79. Total film thickness was 6 mils.

System No. 16, the 100 percent solids epoxy coating on Span 3, was replaced by a laboratory-formulated zinc-silicate paint and vinyl green finish coat, State Specification 61-G-75, in June 1964. The original epoxy system deteriorated so rapidly that severe corrosion was noted within 1 year of its application (Fig. 2).

Photographs of the other test sections were taken in March 1968 after 10 years of exposure (Figs. 3 through 6). The following visual ratings of these test sections were made on October 24, 1968, on a scale of 0 to 10, 10 being perfect condition:

System 14. The post-cure inorganic zinc-silicate-vinyl finish-coat type received an average rating of all test sections of about 9+. Of all systems tried since 1932, this is the best as of the date of this report.

System 15. Average rating of all test sections was about 4.5.

System 16. Rating was 0 after 1 year in 1959. The replacement post-cure type laboratory formula 1R-310, System 23, with a green vinyl finish coat was applied in June 1964 and had a rating of 8. However, this rating of 8 must be qualified because of the extremely adverse conditions at the time the zinc-silicate coating was applied. In scrubbing off the excess curing agent, some of the inorganic zinc-silicate primer was removed from the steel because it did not have sufficient time to cure properly in the foggy weather.

Figure 2. System 16—100 percent solids epoxy coating after 3 years of exposure; photo taken in 1961.

Figure 3. System 14—Inorganic zinc and silicate primer, post-cure type, and vinyl primers and vinyl finish coat.

Figure 4. System 15—Linseed oil-alkyd red lead primer, traffic paint barrier coat, and vinyl primers and vinyl finish coat.

Figure 5. System 17—All-vinyl system.

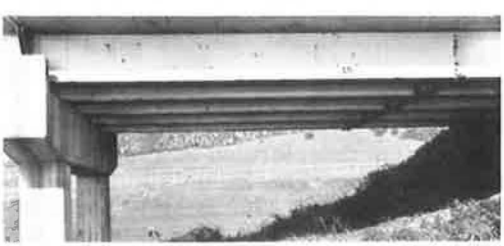

Figure 6. System 18—Linseed oil-alkyd-red lead primer and phenolic iridescent green finish coat.

System 17. Average rating was about 7.0 for all test sections.

System 18. Average rating was 4 to 5.

As noted in the above ratings, after a 10-year exposure the post-cure type inorganic zinc-silicate coating with vinyl finish coat is by a good margin the best coating system tested to date and should provide excellent protection for an additional 10 to 20 years. The all-vinyl system, which ranks second in this series of tests, shows some breakdown on the sharp edges of the upper and lower flanges and appears to be the best of the organic vehicle systems tested.

SAN SIMEON CREEK BRIDGE, 1959 REPAINTING

The following coatings were applied to this bridge in the 1959 repainting. All sandblasted surfaces received a first coat of vinyl wash primer, Federal Specification MIL-P-15328.

System 19. Basic lead silico chromate primer, 4 mils dry film thickness in not less than three applications, was followed by a 2-mil dry film thickness in not less than two applications of phenolic iridescent green, State Specification 58-G-79. This system was applied to the three south spans of the bridge.

System 20. Semiquick-drying red lead primer, State Specification 58-G-53, 3 mils dry film thickness, was applied in not less than two applications over spotblasted areas followed by 1 mil dry film thickness over entire area followed by 2 mils dry film thickness in not less than two applications of phenolic iridescent green, State Specification 58-G-79. This system was applied to all spans north of the three southern spans.

Final inspection in 1964 before repainting this bridge showed that both of these systems were equivalent in corrosion prevention and were rated about 5 on a scale of 0 to 10. It confirmed previously noted observations that the conventional organic vehicles with inhibitive pigments will not provide the protection afforded by certain inorganic zinc-silicate primers in aggressive marine environments.

SAN SIMEON CREEK BRIDGE, 1964 REPAINTING

As noted previously, the post-cure type of inorganic zinc-silicate primer with a vinyl finish coat provided excellent corrosion protection to the steel sections of the Leffingwell Creek Bridge, where this coating system was applied in the 1958 repainting. In the post-cure type of inorganic zinc-silicate coating, zinc dust is dispersed in an aqueous sodium silicate vehicle just before use. The zinc-sodium silicate vehicle is sprayed on the sandblasted steel to provide about a 3-mil dry film thickness coating. After an initial drying period of approximately ½ hour, a solution of phosphoric acid in isopropyl alcohol is sprayed on the coating to cure it. The cure period normally requires about 24 hours, after which the coating becomes very hard and the vehicle becomes insoluble in water.

The curing process results in the deposition of a white salt on the zinc-silicate film that must be removed prior to the application of finish coats. The film deposit is very difficult to remove and requires intense scrubbing with brushes and water. On large complicated steel structures the removal of this film is difficult and therefore costly to achieve by hand-scrubbing.

Self-cure zinc-silicate coatings, which do not require the removal of a film of curing solution products before the application of subsequent paint coats, were known only a few years prior to the repainting of the San Simeon Creek Bridge in 1964. None of the self-cure systems has been exposed to aggressive atmospheres long enough to draw valid conclusions about their performance compared to the post-cure types. The latter have had service histories in various parts of the world approximating 25 to 30 years.

In 1964, the Materials and Research Department wrote a specification, 64-G-55, for a self-cure zinc-silicate paint containing a lithium-sodium silicate vehicle as a result of very successful salt spray tests in the laboratory. In the 1964 repainting of the San Simeon Creek Bridge it was decided to use this material as a primer on the three south spans of this bridge as part of the following system:

System 21. Three mils dry film thickness of self-cure zinc-silicate primer, State Specification 64-G-55, was applied. Following a cure of 48 hours, vinyl wash primer, Federal Specification MIL-P-15328A, was applied, followed by 3 mils dry film thickness of State Specification vinyls 63-G-40, 63-G-41, and 61-G-75. No vinyl wash primer was used on sandblasted steel.

System 22. The remaining spans of the bridge were coated with 6 mils of dry film thickness of the all-vinyl system described in System 21.

When these coatings were evaluated on October 24, 1968, the self-cure inorganic zinc-silicate-vinyl system and the all-vinyl system were rated as 8. The lower rating of the self-cure inorganic zinc-silicate-vinyl system after 4 years compared to the 9+ rating of the post-cure inorganic zinc-silicate-vinyl system after 10 years on the Leffingwell Creek Bridge may be attributed to the following adverse factors present at the time the San Simeon Creek Bridge was repainted in 1964:

1. No agitated spray pot was used. This is absolutely essential for maintaining complete dispersion of the zinc dust pigment at all times.
2. The spray hose was 150 ft long, which was excessive and may have resulted in the settlement of some zinc pigment in the spray hose.
3. The weather was cloudy and misty during application, and heavy rain occurred immediately after application of the self-cure inorganic zinc-silicate primer, which would have inhibited the curing process.

NEW DEVELOPMENTS IN THE INORGANIC ZINC-SILICATE PRIMERS

Following the 1964 repainting of the San Simeon Creek Bridge, research work was done in the laboratory on the modified ethyl silicate vehicles for use in inorganic zinc-silicate coatings. Based on the great promise shown by the ethyl-silicate-type vehicle in extensive salt spray tests, a new specification was written in 1965 that included two types of self-cure inorganic zinc-silicate coatings: Type I, a lithium-sodium silicate vehicle; and Type II, a modified ethyl silicate vehicle.

Type I cures best in a drier atmosphere than is normally experienced on the coast. The cure of Type II, however, is accelerated by high humidities because wet air speeds the hydrolysis of the ethyl silicate vehicle in the curing process. In arid regions it would be mandatory to artificially cure a Type II system by the periodic application of a water spray for 24 to 48 hours.

Mud Creek Bridge, about 30 miles north of San Simeon on Route 1, was coated with the Type II system with vinyl finish coats in 1967. This bridge and other bridges in the coastal zone coated with the Type I and Type II systems will be periodically evaluated in the coming years. Final conclusions as to the merits of the self-cure and post-cure inorganic zinc-silicate systems cannot be made until we have sufficient exposure time with the self-cure systems.

About 3 years ago experiments using high-pressure water jets in the range of 2,000 to 6,000 psi showed that the post-cure reaction products of the post-cure inorganic zinc-silicate coating could be removed efficiently by this procedure. Adoption of this procedure will depend on whether the self-cure systems provide the corrosion protection of the post-cure type.

It should be noted that the solvents in many of the paint systems described in this report do not comply with the air pollution control regulations that first became effective in July 1967 in Los Angeles County and in January 1968 in the Bay Area Counties Air Pollution Control District. A research program, initiated in 1966 and completed prior to July 1, 1967, enabled the California Division of Highways to reformulate all its specification paints to comply with these regulations. The revised formulas for the paints are shown in the January 1969 Standard Specifications of the California Division of Highways.

Evaluation of Welded Deformed Wire Fabric Reinforcement in Machine-Made Concrete Pipe

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A research project was undertaken to evaluate the structural behavior of machine-made concrete pipe reinforced with welded deformed wire fabric and to determine the validity of previously developed design methods for this type of pipe. Test results indicate significantly greater variability of both 0.01-in. crack strength and ultimate strength for machine-made pipe compared to the previously tested cast pipe. As long as this possibility of greater strength variability is recognized, the design formulas previously developed for 0.01-in. crack strength and ultimate flexural strength of cast pipe also apply to machine-made pipe. The design formula for ultimate diagonal tension strength may also be applied to machine-made pipe, but only with a larger safety factor and with certain modifications of provisions tentatively suggested in earlier work. Comparison of test results on companion pipe indicates that deformed wire fabric offers higher 0.01-in. crack strength than smooth wire fabric. For pipe made by the Packerhead process, the degree of improvement is even greater than was previously found for cast pipe.

•SEMI-EMPIRICAL DESIGN METHODS have been developed for concrete pipe with welded deformed wire fabric reinforcements made by the cast process and were previously reported by Heger and Gillespie (4). The general formulation of the equations used in that method, as derived by Heger (1, 2), are based on theoretical reasoning that describes the behavior of the concrete and the steel reinforcement for pipe tested with the three-edge bearing method (ASTM Method C 497). Equations were presented for 0.01-in. crack strength, ultimate flexural strength, and ultimate diagonal tension strength.

Automated machine processes for pipe manufacture are becoming increasingly important in the economical production of larger sizes of pipe. Because the existing design method was derived for cast pipe, the primary goal of this research program was to determine the validity of the theory for these machine-made pipes. A secondary objective of this program was to determine if there is an advantage in using deformed fabric in machine-made pipe beyond that already demonstrated for cast pipe.

TEST PROGRAM

The test program was undertaken to evaluate U.S. Steel welded deformed wire fabric reinforcing in two types of machine-made pipe. This program covers 65 full-size tests on ASTM Specification C 76 pipe ranging in diameter from 48-in. to 96-in. for dry-pack-vibration pipe and 48-in. to 72-in. for Packerhead pipe. Strength classes range from Class II to Class V, with Class V restricted to the 48-in. diameter size. This range of size and strength provides a reasonable spread of the important variables whereby the validity of the design method may be evaluated.

Paper sponsored by Committee on Culverts and Culvert Pipe and presented at the 49th Annual Meeting.

The primary program consisted of three-edge bearing tests on 50 pipes reinforced with deformed fabric, of which 30 were Packerhead pipe and 20 were dry-pack-vibration pipe. The secondary program covered similar tests on 48-in. diameter pipe reinforced with smooth wire fabric; 9 were Packerhead and 6 were dry-pack-vibration pipe.

Reinforcement layout and nominal cover for the test specimens are as shown in Figure 1. Design wall thicknesses are standard ASTM C 76 Wall B. All test specimens were designed to meet ASTM C 76 strength requirements as given in Table A1 (Appendix A). The steel areas called for in the design did not contain an allowance for manufacturing variability, but did include variability factors for design.

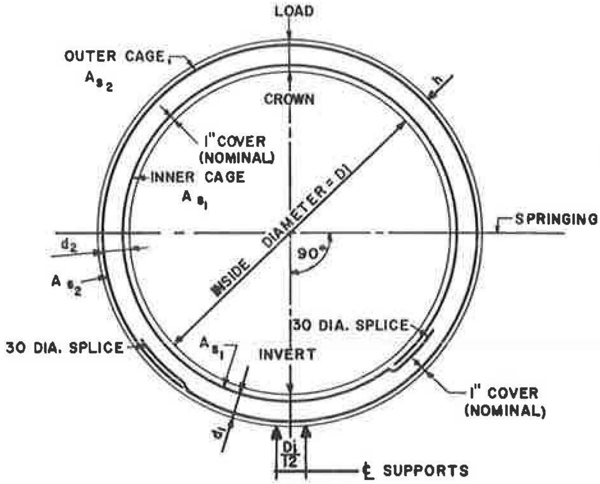

Figure 1. Pipe test specimens—typical transverse section.

For pipe having smooth wire fabric, the "design" steel area was made identical to that used in the companion specimens having deformed fabric. This allows a direct comparison of the relative effects of smooth and deformed wire. Because area requirements for smooth wire are larger than those for deformed wire (whenever 0.01-in. crack strength governs the area of nondeformed 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), the design strength of some pipe specimens with smooth fabric is less than the ASTM C 76 strength class requirements. Therefore, a downward adjustment was made in the specified D-load requirements for these specimens.

Laboratory tests were conducted to determine the significant structural poperties of both the concrete and the steel used in the test specimens. Wire tests were conducted by a commercial testing laboratory. Ultimate tensile strength and yield strength were obtained from representative samples cut from each style of welded wire fabric used in the test program. Wire strength values are given in Table A2 (Appendix A). Wire tests for the 60- and 72-in. Packerhead pipe were also conducted by the Louisiana Department of Highways Laboratory and the results are given in Table 1. They differ substantially from results obtained by the commercial testing laboratory.

TABLE 1

RESULTS OF STATE OF LOUISIANA TESTS ON STEEL SPECIMENS

Pipe Mark	Inner Cage Reinforc.		Outer Cage Reinforc.	
	Furnished A_{s1} (in²./ft)	Ultimate Strength (psi)	Furnished A_{s1} (in²./ft)	Ultimate Strength (psi)
PH 60-2 a, b, c	0.192	118,750	0.156	96,154
PH 60-3 a, b, c	0.265	111,111	0.192	118,750
PH 60-4 a, b, c	0.851	95,205	0.331	126,816
PH 72-2 a, b, c	0.228	100,000	0.192	112,500
PH 72-3 a, b, c	0.331	126,816	0.228	100,000
PH 72-4 a, b, c	0.984	92,683	0.444	94,595

Concrete mixes were designed by the individual pipe manufacturer to meet the design strength as closely as possible. Concrete samples were tested at laboratories selected by the individual manufacturer. Compression tests were conducted on two 4-in. diameter concrete cores removed from the walls of each test specimen. Cores were cut in the region of the quarter points of the ring. The compressive strength for the concrete was taken as 0.85 times the core strength (the core strength being the test value corrected for length-diameter ratio in accordance with ASTM Method C 42). This reduction is based on correlations between the core and cylinder strength determined in the earlier test programs, and the design equations are based on this interpretation of core test data.

Test pipes were loaded in three-edge bearing at the plants of the companies participating in the program, following the requirements of ASTM Method C 497. All testing machines used in the program were calibrated by an independent testing agency in the 6 months prior to testing of the pipe.

Test specimens were loaded to failure at an approximate rate of 2,000 lb per foot of length per minute or 16,000 lb per minute. The load was recorded at the occurrence of the first visible crack and at the 0.01-in. crack. The load at which each crack occurred, principally in the crown and invert, was noted, and the number, spacing, and pattern of these cracks were recorded. Finally, the failure load and the mode of failure were recorded.

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 and invert and at the core locations.

TEST RESULTS

Principal test results are given in Table A3 (Appendix A). Strength results are given in terms of D-load strength. D-load strength is defined as the test load per foot of pipe length divided by the nominal inside diameter of the pipe in feet. A comparison of "test D-load" and "design D-load" is also given in Table A3. The design D-load is the D-load strength determined by the design procedure previously developed for cast pipe and used in proportioning the test specimens (2, 4). The steel areas called for in the design do not contain an allowance for manufacturing variability, but do include a variability factor for design.

Table A4 (Appendix A) gives the pipe deflections measured during testing. Deflections are given for both the measured 0.01-in. crack load and at 1.4 times the ASTM C 76 0.01-in. crack load. The latter reading was selected to provide a reference point at a fixed load for each pipe class near to but prior to the ultimate load capacity of the pipe.

THEORY OF STRUCTURAL BEHAVIOR

The semi-empirical equations used in the present study are the same as those developed by Simpson Gumpertz and Heger, Inc., in the previous study of wet-cast concrete pipe reinforced with U.S. Steel welded deformed wire fabric (2, 4).

The term, $C_L N_L$, included as a tentative term in the diagonal tension strength equation proposed by Heger and Gillespie (4, Eq. 17; also Eq. 4.18 in 2), is dropped. The term provided a small increase in the calculated diagonal tension strength when fabric longitudinals were spaced at 8 in. on center or less. Originally it was included on a tentative basis as a possible explanation of higher diagonal tension strengths in certain earlier tests. However, the results of the present program do not substantiate the increased diagonal tension strength indicated by this term.

VARIABILITY FACTORS FOR DESIGN AND MANUFACTURING

As has been noted, the design equations are based in part on theory and in part on the results of full-scale pipe tests. The variables in the equations and their form were established by theoretical analysis and physical reasoning describing the behavior of pipe under three-edge bearing loads. Then, certain constants used in the equations were determined from the average of a large number of full-scale test results. When these equations are used for design of production pipes, modifications must be incorporated to allow for variations between the test strength of a given pipe and the calculated average test strength.

The sources of variability are placed in two main categories in an attempt to separate the variations that apply to the direct evaluation of test data from the variations arising during the manufacture of production pipe. These categories may be defined as follows:

1. Variability of theory itself—Evaluation of test data is subject to many sources of variation. Test results with reinforced concrete structural components inevitably show scatter, particularly with respect to flexural crack widths and diagonal tension strength. Further scatter is caused by inaccuracies in the measurement of material properties, dimensions, and steel location and by nonuniformity in these quantities throughout the test specimen. These variations are associated with the inability of the theory to predict the true test strength of specimens with measured dimensional and physical parameters. They are accounted for by the introduction of the variability coefficients $\phi_{0.01}$, ϕ_f, and ϕ_d, as explained later in this section.
2. Variations inherent in the manufacture of production pipe—Properties of materials may be nonuniform and may differ from design requirements (i.e., steel placement and wall dimensions will vary in production pipe). Furthermore, specific processes may introduce further variations. The magnitude of such variations is affected by the particular process, process control, materials characteristics, and degree of quality control at individual plants. This variation is accounted for by the introduction of a variability coefficient, ϕ_x.

The design procedure suggested elsewhere (2, 4) for cast pipe with deformed wire fabric utilizes equations with suggested design variability coefficients, $\phi_{0.01}$, ϕ_f, and ϕ_d, that allow for the variability of the theory itself, as indicated by correlation with control tests. These design variability coefficients (see notation in Appendix B) are applied directly to the calculated average D-load to obtain the minimum required D-load that would be adequate if the pipe could be produced with all parameters as assumed in the design (i.e., with steel placement and wall thickness exactly in accordance with the design and with construction free from variations caused by particular process or material characteristics). The variations resulting from process, plant practice, and local materials are accounted for in the design by increasing the calculated steel area sufficiently to allow for their effects. The following summarizes the suggested design approach:

1. Use equations given by Heger (2, 4) to determine minimum circumferential steel areas and other design requirements. Increase the specified requirements for 0.01-in. crack strength, ultimate flexural strength, and ultimate diagonal tension strength to allow for variability of design theory. Modify D-loads as follows:

$$\text{modified}\left(DL_{0.01} + \frac{9W}{D_i}\right) = \frac{DL_{0.01} + \dfrac{9W}{D_i}}{\phi_{0.01}}$$

$$\text{modified}\left(DL_u + \frac{6W}{D_i}\right) = \frac{DL_u + \dfrac{6W}{D_i}}{\phi_f}$$

(ultimate flexure)

$$\text{modified}\left(DL_u + \frac{11W}{D_i}\right) = \frac{DL_u + \dfrac{11W}{D_i}}{\phi_d}$$

(ultimate diagonal tension)

where $\phi_{0.01}$, ϕ_f, and ϕ_d are design variability coefficients less than 1.0. The following values are suggested by Heger (2): $\phi_{0.01} = 0.90$, $\phi_f = 0.95$, and $\phi_d = 0.90$. These are based on the correlation of theory and test data for cast pipe given elsewhere (2, 4). The terms involving W/D_i are the D-load equivalents of the pipe weight for the different strength criteria.

2. Increase the steel area necessary to obtain the modified strengths given in step 1 above to allow for variability inherent in the manufacturing process:

$$A_s \text{ production} = \frac{A_s}{\phi_x}$$

calculated based on D-loads in step 1 and with ϕ_x a manufacturing variability coefficient less than 1.0. No recommendation is provided for specific values of ϕ_x. These must be determined by individual pipe producers based on their own local conditions of material and process characteristics and quality control.

In the case of machine-made pipe, certain types of variability due to the process occur that do not clearly fit in either of the preceding categories. Variability caused by process characteristics that affect the local integrity or uniformity of the concrete or its bond with the steel, or both, is related to the inherent applicability of the design theory. Variability may also be caused by process characteristics that affect the accuracy of steel placement and the average strength properties of the concrete; this fits into the second category.

Because both of these types of variation probably depend more on local plant conditions than on inherent process characteristics, they should probably be accounted for in the manufacturing variability coefficient, ϕ_x. However, it should be noted that, because the manufacturing variability coefficient, ϕ_x, is applied only to the steel area (in contrast to $\phi_{0.01}$, ϕ_f, and ϕ_d, which are applied directly to the D-load strength), ϕ_x may have a larger range of values between different plants, processes, and localities for designs governed by ultimate diagonal tension strength than for designs governed by 0.01-in. crack strength or ultimate flexural strength. This occurs because, as shown by the equations previously given, the circumferential steel area provides a relatively smaller influence on diagonal tension strength than on the other strength criteria.

The correlation between the theory and the test results for machine-made pipe indicates that in some cases process-induced variations may require increased allowance for variability when the equations of Heger and Gillespie (2, 4) are used for design of machine-made pipe.

CORRELATION OF THEORY AND TESTS

0.01-in. Crack Strength

Test and calculated $DL_{0.01}$ values and the ratio of test to calculated values for each test specimen are compared in Table A5 (Appendix A). In all cases calculations were based on measured values of wall thickness, concrete cover thickness, steel area, and concrete strength of each test specimen.

Statistical parameters that compare the test and calculated strengths for the present program, as well as for two previous programs covering pipe made by the wet-cast process (2, 3), are given in Table A6 (Appendix A). Average values of the ratio of the test to calculated $DL_{0.01}$ and the coefficient of variation of this ratio are presented for selected groupings of test pipe. The particular groupings are selected in order to compare design parameters or process characteristics that might affect the applicability of the design equations.

Correlation of test results and the 0.01-in. crack strength theory is reasonably good for the entire group of pipe having deformed fabric reinforcement and produced by the Packerhead and the dry-pack-vibration processes. However, the variability of results for Packerhead pipe in this test program appeared somewhat higher than for either dry-pack-vibration or previously tested cast pipe. This is due primarily to the low ratio of test strength to calculated strength for Classes IV and V and the high ratio of test strength to calculated strength for Class II Packerhead pipe.

The 0.01-in. crack strength test results for dry-pack-vibration pipe reinforced with deformed fabric showed about the same variability as exhibited by the cast pipe in the previous program. The ratio of test to calculated 0.01-in. crack strength for every group of Packerhead pipe with smooth fabric is less than that for companion pipe with deformed wire fabric. This indicates that deformed wire fabric is more effective than smooth wire fabric in limiting the effects of variables in the Packerhead process that reduce the 0.01-in. crack strength. Voids, which were observed adjacent to reinforcing in pipe made by both processes, may lower the bond strength. This may increase the slip between steel and concrete and result in an increased crack width. Wire deformations minimize the loss of bond due to voids.

Ultimate Strength

Test values and calculated DL_u values and the ratio of test values to calculated values for each test specimen are given in Table A7 (Appendix A). Calculations are based on actual measured values of wall thickness, concrete cover thickness, steel area, concrete strength, and steel strength. For those test pipes having nearly the same calculated DL_u values for both flexural and diagonal tension failure, both values are given in Table A7. The lower value of the calculated flexural or diagonal tension DL_u produces the higher ratio of test to calculated DL_u, and this higher ratio is the one used for the correlation of test and theory.

Statistical parameters comparing the test strengths and calculated strengths for the present program, as well as for the previous program covering pipes made by the wet-cast process (2, 4), are given in Table A8 (Appendix A). Average values of the ratio of test to calculated DL_u and the coefficient of variation of this ratio are presented for selected groupings of test pipes.

The correlation of test results and theory is reasonably good for ultimate flexural strength of both the dry-pack-vibration pipe and the Packerhead pipe with deformed reinforcement. However, the variability of results for Packerhead pipe is somewhat higher than either dry-pack-vibration or cast pipe. Computed strengths of these pipes were based on the ultimate strength of the wires as determined by a commercial testing laboratory. The ultimate strengths of the wires as obtained by the Louisiana Department of Highways Laboratory were 15 to 20 percent higher for those pipes tested at Baton Rouge. If these values are used in the calculations, the ratio of the test strength to calculated ultimate flexural strength would be close to one. Thus, the true variability of flexural strength may not be as high for Packerhead pipe as that given in Table A8.

The correlation of test results and theory is reasonably good for ultimate flexural strength of dry-pack-vibration pipe with welded smooth wire fabric reinforcement. The Packerhead pipe with smooth wire does not reach the flexural strength indicated by the calculations, whereas the companion pipe with deformed wire reached or exceeded the calculated value. This indicates that the Packerhead process causes a lowering of the ultimate flexural strength for pipe with smooth wire fabric but not for pipe with deformed fabric. This may be caused by slippage between the steel and the concrete, but this is not readily evident in the limited test data.

The correlation of test results and the theory for diagonal tension failure previously developed for cast pipe is reasonably good for Packerhead pipe with deformed fabric. Class V Packerhead pipe is an exception to this, however, because test results are significantly lower than the calculated results. The variability of the ratio of the test strength to calculated strength for Packerhead pipe is higher than that obtained for either dry-pack-vibration pipe or cast pipe. The higher variability reflects operator or process variations or other effects not accounted for by the design equation.

For dry-pack-vibration pipes with welded deformed wire fabric, the diagonal tension strengths obtained in the tests were consistently lower than the calculated values. A number of the test specimens had visible circumferential cracks at their ends prior to loading. Such cracks could increase slabbing tendencies and result in a reduction in radial and diagonal tension strength. The available test data are not extensive enough to indicate whether this is an inherent characteristic of the process or an individual plant problem.

For pipes with smooth wire fabric reinforcement, ultimate diagonal tension strength test results are lower than the calculated results for both Packerhead and dry-pack-vibration pipes. Test results for these pipes are also lower than the results of companion pipes with welded deformed wire fabric reinforcement. This indicates that, for both processes, deformed wire fabric is more effective than smooth wire fabric in limiting the effects of variables that reduce the ultimate diagonal tension strength.

CONCLUSIONS

The test results and analysis presented indicate that design equations previously developed for cast pipes with deformed fabric reinforcement (2, 4) can also be used for Packerhead and dry-pack-vibration pipes. However, in order to utilize this design method for pipes made by these processes, an adjustment may be necessary in some of the factors for design variability that had been previously suggested for cast pipe ($\phi_{0.01}$, ϕ_d, ϕ_f) or in the factor for manufacturing variability, ϕ_x. The significance of these variability factors in the application of the Heger and Gillespie design method (2, 4) is explained in the section on variability factors for design and manufacturing. The design variability coefficients suggested for general use must be verified and coordinated with a properly selected manufacturing variability factor, based on tests at individual plants, before the design method can be used for final design at those plants.

The increase in ultimate diagonal tension strength indicated by the term $C_L N_L$ (Eq. 4.18 in 2; Eq. 17 in 4) is not substantiated by the present tests, and therefore the term is dropped. Further tests are required to determine whether this term should be retained for cast pipe.

The comparative tests between 48-in. diameter Packerhead and dry-pack-vibration pipes with welded deformed wire fabric reinforcement and similar pipes with welded smooth wire fabric reinforcing indicate substantially higher 0.01-in. crack strength for the pipes with deformed wire fabric. A further comparison with previous results obtained for cast pipes with both types of reinforcing indicates that, although deformed wire fabric provides significant improvement in 0.01-in. crack strength performance for both machine-made pipe and for cast pipe, the degree of improvement is often greater in machine-made pipe than in cast pipe.

The comparative results also show somewhat greater diagonal tension ultimate strengths for both the Packerhead and the dry-pack-vibration pipes with deformed wire fabric and greater flexural ultimate strength for the Packerhead pipes with deformed wire fabric.

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Appendix A

DATA ON STEEL PIPE TEST SPECIMENS

TABLE A1

DESIGN REQUIREMENT FOR TEST SPECIMENS

	Mark		Type of Pipe			Inner Cage			Outer Cage		
	Packerhead Process Pipe (PH)	Dry-Pack Vibration Process Pipe (DV)	Internal Diam. of Pipe in.	Wall B Thickness in.	ASTM Class	Min. Design Area in^2/ft	Furnished Size	Nominal Furnished Area in^2/ft	Min. Design Area in^2/ft	Furnished Size	Nominal Furnished Area in^2/ft
Deformed Wire	PH 48-2 a,b,c	DV 48-2 a,b	48	5	II	0.13	D2.4/7	0.14	0.09	D2.4/7	0.14
	PH 48-3 a,b,c	DV 48-3 a,b			III	0.17	D2.8/7	0.17	0.12	D2.4/7	0.14
	PH 48-4 a,b,c	DV 48-4 a,b			IV	0.48	D8/6	0.48	0.25	D4.4/7	0.26
	PH 48-5 a,b,c	DV 48-5 a,b			V	0.91	D15.2/4	0.91	0.50	D8.4/6	0.50
	PH 60-2 a,b,c		60	6	II	0.19	D3.2/7	0.19	0.14	D2.4/7	0.14
	PH 60-3 a,b,c				III	0.26	D4.4/7	0.26	0.20	D3.2/7	0.19
	PH 60-4 a,b,c				IV	0.73	D12.2/5	0.73	0.34	D5.5/7	0.33
	PH 72-2 a,b,c	DV 72-2 a,b	72	7	II	0.23	D3.8/7	0.23	0.17	D2.8/7*	0.17
	PH 72-3 a,b,c	DV 72-3 a,b			III	0.32	D5.5/7	0.33	0.23	D3.8/7	0.23
	PH 72-4 a,b,c	DV 72-4 a,b			IV	0.98	D16.4/4	0.98	0.44	D7.4/6†	0.44
		DV 96-2 a,b	96	9	II	0.32	D5.5/7	0.33	0.24	D3.8/7	0.23
		DV 96-3 a,b			III	0.51	D8.4/6	0.51	0.34	D5.5/6	0.33
		DV 96-4 a,b ††			IV	0.84	D14/5	0.84	0.63	D10.5/5	0.63
Smooth Wire	PH 48-2P a,b,c	DV 48-21 a,b	48	5	II A ø	0.13	7¼/8	0.14	0.09	7¼/8	0.14
	PH 48-3P a,b,c	DV 48-3P a,b			III A	0.17	6 /8	0.17	0.12	7¼/8	0.14
	PH 48-4P a,b,c	DV 48-4P a,b			IV	0.48	00½/6	0.48	0.25	3½/8	0.26

Notes:

1. All Pipe Wall B
2. Pipe Length 8'-0", Production - Tongue and Groove Ends
3. Design Concrete Strength - 5000 psi
4. Wire Spacing - Circumferentials: 2 inches o.c.
   Longitudinals: 8 inches o.c.

* For DV72-2 a,b,: D2.6/7, Area = 0.16 in^2 was furnished

† For DV72-4 a,b,: D8.4/6, Area = 0.50 was furnished

†† DV 96-4 a,b required stirrups

ø A indicates C76 strength class is adjusted to account for method of proportioning specimens having smooth wire. See Section 2-2.

TABLE A2

DESCRIPTION OF TEST SAMPLES

Fabric	Pipe Mark	ASTM Class [6]	Nom. I.D. In.	Wall Thickness: Crown in.	Wall Thickness: Springing Left in.	Wall Thickness: Springing Right in.	Wall Thickness: Invert in.	Inner Cage: Wire Size [5] # or in.	Inner Cage: Furnished A_s in²/ft [1]	Inner Cage: Steel Test Props. Ult. St. psi [1]	Inner Cage: Yield St. psi	Inner Cage: Effective d_s Crown in.	Inner Cage: Effective d_s Invert in.	Outer Cage: Wire Size [5] # or in.	Outer Cage: Furnished A_{s_2} in²/ft [1]	Outer Cage: Steel Ult. St. by Test psi [1]	Outer Cage: Effective d_s At Springing Left in.	Outer Cage: Effective d_s At Springing Right in.	Concrete: Avg. Comp. Str. of Core (Corrected) psi [2]	Condition of Reinforcement [3]	Pipe Mark
DEFORMED WIRE FABRIC	DV 48-2 a*	II	48"	5.25	5.13	5.19	5.19	D2.4/7	.145	91280	85275	3.28	3.72	D2.4/7	.145	91280	4.04	3.79	5561	L.R.	DV 48-2 a
	DV 48-2 b	II	48"	5.06	5.06	5.13	5.13	D2.4/7	.145	91280	85275	4.09	4.29	D2.4/7	.145	91280	4.09	4.16	4675	L.R.	DV 48-2 b
	PH 48-2 a†	II	48"	5.19	5.13	5.13	5.19	D2.4/7	.145	88320	83875	3.91	3.79	D2.4/7	.145	88320	3.85	3.60	4740	B.	PH 48-2 a
	PH 48-2 b	II	48"	5.13	5.13	5.13	5.13	D2.4/7	.145	88320	83875	3.66	3.60	D2.4/7	.145	88320	3.98	3.98	6726	B.	PH 48-2 b
	PH 48-2 c	II	48"	5.25	5.13	5.31	5.25	D2.4/7	.145	88320	83875	3.85	3.91	D2.4/7	.145	88320	3.60	4.03	6316	B.	PH 48-2 c
	DV 48-3 a*	III	48"	5.25	5.13	5.19	5.13	D2.8/7	.173	85080	79100	4.53	4.29	D2.4/7	.145	91260	4.10	3.85	5346	L.R.	DV 48-3 a
	DV 48-3 b	III	48"	5.19	5.06	5.19	5.13	D2.8/7	.173	85080	79100	4.41	4.10	D2.4/7	.145	91260	3.59	4.35	5672	L.R.	DV 48-3 b
	PH 48-3 a†	III	48"	5.31	5.25	5.38	5.19	D2.8/7	.172	86880	81300	3.97	3.72	D2.4/7	.145	88320	4.28	4.16	5891	B.	PH 48-3 a
	PH 48-3 b	III	48"	5.25	5.06	5.25	5.19	D2.8/7	.172	86880	81300	3.97	3.54	D2.4/7	.145	88320	3.91	3.97	4448	B.	PH 48-3 b
	PH 48-3 c	III	48"	5.13	5.13	5.13	5.19	D2.8/7	.172	86880	81300	3.79	3.66	D2.4/7	.145	88320	3.64	3.91	4820	B.	PH 48-3 c
	DV 48-4 a*	IV	48"	5.13	5.06	5.13	5.13	D8/6	.475	85080	77225	3.72	4.16	D4.4/7	.261	91500	3.94	3.82	5453	L.R.	DV 48-4 a
	DV 48-4 b	IV	48"	5.13	5.13	5.13	5.13	D8/6	.475	85080	77225	4.34	4.22	D4.4/7	.261	91500	3.82	4.01	4444	L.R.	DV 48-4 b
	PH 48-4 a†	IV	48"	5.13	5.13	5.00	5.13	D8/6	.475	83080	74375	3.72	3.72	D4.4/7	.264	87180	3.76	3.57	5384	B.	PH 48-4 a
	PH 48-4 b	IV	48"	5.31	5.25	5.00	5.19	D8/6	.475	83080	74375	3.84	3.65	D4.4/7	.264	87180	3.68	3.75	5511	B.	PH 48-4 b
	PH 48-4 c	IV	48"	5.13	5.19	5.06	5.00	D8/6	.475	83080	74375	3.66	3.40	D4.4/7	.264	87180	3.69	3.63	5227	B	PH 48-4 c
	DV 48-5 a*	V	48"	5.13	5.13	5.13	5.13	D15.2/4	.915	90320	75400	4.16	4.10	D8.4/6	.520	98850	4.16	3.34	5895	L.R.	DV 48-5 a
	DV 48-5 b	V	48"	5.19	5.13	5.19	5.06	D15.2/4	.915	90320	75400	3.72	3.59	D8.4/6	.520	98850	3.59	3.40	5177	L.R.	DV 48-5 b
	PH 48-5 a†	V	48"	5.06	5.13	5.06	5.13	D15.2/4	.924	82150	76300	3.71	3.47	D8.4/6	.514	91950	3.91	3.90	5246	L.R.	PH 48-5 a
	PH 48-5 b	V	48"	5.13	5.13	5.19	5.19	D15.2/4	.924	82150	76300	3.66	4.09	D8.4/6	.514	91950	3.97	3.78	5703	L.R.	PH 48-5 b
	PH 48-5 c	V	48"	5.00	5.19	5.19	5.25	D15.2/4	.924	82150	76300	3.40	4.15	D8.4/6	.514	91950	3.15	4.59	6338	L.R.	PH 48-5 c
	PH 60-2 a	II	60"	6.06	6.19	6.31	6.31	D3.2/7	.188	84880	75475	4.21	4.46	D2.4/7	.146	89850	4.35	4.84	5612	B., S.R.	PH 60-2 a
	PH 60-2 b	II	60"	5.94	6.31	6.06	6.13	D3.2/7	.188	84880	75475	3.53	4.09	D2.4/7	.146	89850	4.59	4.59	4319	B., S.R.	PH 60-2 b
	PH 60-2 c	II	60"	6.06	6.63	6.19	6.31	D3.2/7	.188	84880	75475	3.90	4.65	D2.4/7	.146	89850	5.04	4.85	2579	B., S.R.	PH 60-2 c
	PH 60-3 a	III	60"	6.50	6.44	6.19	6.31	D4.4/7	.260	90980	84750	4.19	4.56	D3.2/7	.188	84880	4.84	4.53	3992	B., S.R.	PH 60-3 a
	PH 60-3 b	III	60"	6.13	6.19	6.06	6.13	D4.4/7	.260	90980	84750	3.95	4.26	D3.2/7	.188	84880	4.59	4.33	3535	B., S.R.	PH 60-3 b
	PH 60-3 c	III	60"	6.25	6.31	6.13	6.06	D4.4/7	.260	90980	84750	4.13	3.81	D3.2/7	.188	84880	4.77	4.47	5926	B., S.R.	PH 60-3 c
	PH 60-4 a	IV	60"	6.13	6.06	6.38	6.19	D12.5/5	.724	93250	85575	4.30	4.61	D5.5/7	.361	82220	4.18	5.06	4180	B., S.R.	PH 60-4 a
	PH 60-4 b	IV	60"	6.06	6.13	6.06	6.13	D12.5/5	.724	93250	85575	4.30	4.55	D5.5/7	.361	82220	4.37	4.37	4051	B., S.R.	PH 60-4 b
	PH 60-4 c	IV	60"	6.19	6.06	6.25	6.13	D12.5/5	.724	93250	85575	4.86	4.62	D5.5/7	.361	82220	4.68	4.62	5453	B., S.R.	PH 60-4 c
	DV 72-2 a	II	72"	7.06	6.88	6.88	7.38	D3.8/ [4]	.205	100280	97650	5.32	6.21	D2.6/7	.175	86100	5.73	5.85	6583	B.	DV 72-2 a
	DV 72-2 b	II	72"	7.44	6.88	6.94	7.19	D3.8/	.205	100280	97650	5.33	5.89	D2.6/7	.175	86100	5.66	5.85	5597	B.	DV 72-2 b
	PH 72-2 a	II	72"	7.00	7.13	7.06	7.13	D3.8/7	.206	85120	78525	5.33	5.89	D2.8/7	.170	87650	5.29	5.41	4281	B., L.R.	PH 72-2 a
	PH 72-2 b	II	72"	7.13	7.06	7.13	7.06	D3.8/7	.206	85120	78525	5.83	5.57	D2.8/7	.170	87650	5.41	4.91	3517	B., S.R.	PH 72-2 b
	PH 72-2 c	II	72"	7.06	7.13	7.00	7.00	D3.8/7	.206	85120	78525	5.95	5.83	D2.8/7	.170	87650	4.98	4.97	3223	B., S.R.	PH 72-2 c
	DV 72-3 a	III	72"	7.25	7.00	7.13	7.06	D5.5/	.347	90450	87350	5.74	5.49	D3.8/	.205	100280	5.83	5.77	5326	B.	DV 72-3 a
	DV 72-3 b	III	72"	7.00	7.31	7.06	7.00	D5.5/	.342	90450	87350	6.24	6.06	D3.8/	.205	100280	6.64	6.20	6230	B.	DV 72-3 b
	PH 72-3 a	III	72"	7.25	7.19	7.19	7.06	D5.5/7	.361	82220	75850	5.93	5.68	D3.8/7	.206	85120	4.89	5.20	3531	B., S.R.	PH 72-3 a
	PH 72-3 b	III	72"	7.13	7.13	7.06	7.19	D5.5/7	.361	82220	75850	5.94	6.00	D3.8/7	.206	85120	5.08	4.82	2911	B., S.R.	PH 72-3 b
	PH 72-3 c	III	72"	7.13	7.13	7.13	7.19	D5.5/7	.361	82220	75850	5.56	5.93	D3.8/7	.206	85120	5.14	5.39	4813	B., S.R.	PH 72-3 c
	DV 72-4 a	IV	72"	7.06	7.00	7.00	7.25	D16.4/4	.979	82580	76125	5.83	5.71	D8.4/	.497	96850	6.15	6.09	6316	B.	DV 72-4 a
	DV 72-4 b	IV	72"	7.13	7.06	6.94	7.06	D16.4/4	.979	82580	76125	5.96	5.83	D8.4/	.497	96850	6.46	5.97	4913	B.	DV 72-4 b
	PH 72-4 a	IV	72"	7.19	7.06	7.00	7.13	D16.4/4	.981	83020	77525	5.77	5.52	D7.4/6	.411	101180	5.03	5.16	6363	B., S.R.	PH 72-4 a
	PH 72-4 b	IV	72"	7.00	7.06	7.00	7.00	D16.4/4	.981	83020	77525	5.46	5.39	D7.4/6	.411	101180	5.41	4.60	6406	B., S.R.	PH 72-4 b
	PH 72-4 c	IV	72"	7.13	7.13	7.13	7.13	D16.4/4	.981	83020	77525	5.46	5.21	D7.4/6	.411	101180	4.85	5.04	5747	B., S.R.	PH 72-4 c
	DV 96-2 a	II	96"	9.31	9.13	9.06	9.00	D5.5/	.342	90450	87350	8.18	7.43	D3.8/	.205	100280	7.64	7.70	5458	B.	DV 96-2 a
	DV 96-2 b	II	96"	9.06	9.00	9.19	8.81	D5.5/	.342	90450	87350	7.93	7.30	D3.8/	.205	100280	7.83	7.77	6738	B.	DV 96-2 b
	DV 96-3 a	III	96"	9.06	9.19	8.94	9.06	D8.4/	.497	96850	83725	7.40	7.59	D5.5/	.342	90450	7.93	7.18	6196	B.	DV 96-3 a
	DV 96-3 b	III	96"	9.00	9.25	9.13	9.38	D8.4/	.497	96850	83725	6.90	8.03	D5.5/	.342	90450	7.49	7.31	5885	B.	DV 96-3 b
	DV 96-4 a**	IV	96"	9.25	9.00	9.00	9.13	D14.0/5	.850	96580	88625	7.60	7.42	D10.5/5	.624	86050	7.44	7.32	6769	B.	DV 96-4 a
	DV 96-4 b	IV	96"	9.50	9.06	9.06	9.31	D14.0/5	.850	96580	88625	8.29	7.47	D10.5/5	.624	86050	7.19	7.19	6331	B.	DV 96-4 b
SMOOTH WIRE FABRIC	DV 48-2P a*	II A [7]	48"	5.19	5.00	5.06	5.13	7¼/8	.139	90250	82775	4.10	4.04	7¼/8	.139	90250	3.91	4.03	5096	L.R.	DV 48-2P a
	DV 48-2P b	II A	48"	5.19	5.06	5.13	5.13	7¼/8	.139	90250	82775	4.47	3.60	7¼/8	.139	90250	3.66	3.66	5315	L.R.	DV 48-2P b
	PH 48-2P a†	II A	48"	5.13	5.06	5.25	5.25	7¼/8	.139	92100	79000	3.73	3.91	7¼/8	.139	92100	3.34	3.91	6266	B.	PH 48-2P a
	PH 48-2P b	II A	48"	5.13	5.25	5.19	5.19	7¼/8	.139	92100	79000	3.60	3.97	7¼/8	.139	92100	3.85	3.91	7352	B.	PH 48-2P b
	PH 48-2P c	II A	48"	5.19	5.25	5.13	5.19	7¼/8	.139	92100	79000	3.72	3.79	7¼/8	.139	92100	3.41	3.66	5673	B.	PH 48-2P c
	DV 48-3P a*	III A	48"	5.19	5.13	5.06	5.19	6 /8	.172	81250	78300	3.90	4.15	7¼/8	.139	90250	4.16	3.72	5227	L.R.	DV 48-3P a
	DV 48-3P b	III A	48"	5.25	5.13	5.13	5.13	6 /8	.172	81250	78300	4.34	4.03	7¼/8	.139	90250	4.04	3.60	5020	L.R.	DV 48-3P b
	PH 48-3P a†	III A	48"	5.13	5.13	5.13	5.00	6 /8	.181	85080	78075	3.59	3.15	7¼/8	.139	92100	3.85	3.73	4916	B.	PH 48-3P a
	PH 48-3P b	III A	48"	5.06	5.25	5.19	5.25	6 /8	.181	85080	78075	3.58	3.65	7¼/8	.139	92100	4.16	3.72	5767		PH 48-3P b
	PH 48-3P c	III A	48"	5.13	5.00	5.13	5.06	6 /8	.181	85080	78075	3.78	3.52	7¼/8	.139	92100	3.41	3.73	6143		PH 48-3P c
	DV 48-4P a*	IV	48"	5.06	5.06	5.06	5.19	00½/6	.480	84400	75750	4.34	4.22	3½/8	.257	87780	2.95	3.70	5484	L.R.	DV 48-4P a
	DV 48-4P b	IV	48"	5.19	5.13	5.13	5.25	00½/6	.480	84400	75750	4.15	4.53	3½/8	.257	87780	2.96	4.08	5469	L.R.	DV 48-4P b
	PH 48-4P a†	IV	48"	5.13	5.25	5.31	5.25	00½/6	.480	85780	74780	3.72	3.90	3½/8	.257	84320	3.89	4.07	5945	B.	PH 48-4P a
	PH 48-4P b	IV	48"	5.06	5.19	5.19	5.06	00½/6	.480	85780	74780	3.77	3.84	3½/8	.257	84320	3.95	3.95	3568	B.	PH 48-4P b
	PH 48-4P c	IV	48"	5.13	5.13	5.13	5.13	00½/6	.480	85780	74780	3.84	3.66	3½/8	.257	84320	3.52	3.52	5039	B.	PH 48-4P c

NOTES

1 Steel areas and ultimate strength measured in laboratory tests
2 Corrected core strength is 0.85 x test strength
3 Condition of Reinforcement-
  L.R. - Light Rust
  S.R. - Spotty Rust
  B. - Bright
4 Longitudinal wire size not shown indicates data not recorded
5 Wire Spacing -
  Circumferential: 2 inches o.c.
  Longitudinal: 8 inches o.c.
6 All pipe Wall B
7 A indicates adjusted strength class; see Section 2-2

* Dry Vibrated pipe in 48 inch size had Tongue and Groove ends with Plain Tongue

† Packerhead pipe in 48 inch size had Tongue and Groove ends with Improved Bell

** DV-96-4 a, b pipe had 10 stirrups ASTM A-15 (305) est. 3/8" dia., 4" o.c. Long., 6" o.c. circ. for 4a and 8" o.c. for 4b

CURE CONDITIONS

DV - 48 and DV - 48P series: 3 hour water spray; 12 hours 120°F steam; 24 hour water spray; yard storage

PH - 48 and PH - 48P series: 3 hour water spray; 12 hours 120°F steam in kiln; 24 hours cooling in kiln; yard storage

PH - 60 and PH - 72 series: air cure 3 hours; 24 hours water spray 75°F ± 50°F → 80°F; yard storage

DV - 72 and DV - 96 series; 12-14 hours steam at 110-130°F

TABLE A3

SUMMARY OF TEST RESULTS

Pipe Mark	1st Visible Crack D-Load	D-LOAD 0.01" CRACK Required[4]	D-LOAD 0.01" CRACK Test	$\frac{DL_{.01\ test}}{DL_{.01\ required}}$	D-LOAD ULTIMATE Required[4]	D-LOAD ULTIMATE Test	$\frac{DL_{u\ test}}{DL_{u\ required}}$	Mode of Failure (See Key Below) Test
DEFORMED WIRE FABRIC								
DV 48-2 a	969	1000	1281	1.28	1500	2281	1.52	F.
DV 48-2 b	875	1000	1688	1.69	1500	2438	1.63	F., R. T.
PH 48-2 a	688	1000	1344	1.34	1500	2281	1.52	F., D. T.
PH 48-2 b	906	1000	1719	1.72	1500	2406	1.60	F.
PH 48-2 c	813	1000	1438	1.44	1500	2219	1.48	F.
DV 48-3 a	1000	1350	1500	1.11	2000	2906	1.45	F., D. T.
DV 48-3 b	938	1350	1500	1.11	2000	2813	1.41	F.
PH 48-3 a	813	1350	1656	1.23	2000	2625	1.31	F., R. T.
PH 48-3 b	938	1350	1500	1.11	2000	2563	1.28	F.
PH 48-3 c	969	1350	1375	1.02	2000	2500	1.25	F., D. T.
DV 48-4 a	1188	2000	1813	0.91	3000	3000	1.00	D. T.
DV 48-4 b	1844	2000	3219+[2]	1.61+	3000	3219	1.07	D. T.
PH 48-4 a	1156	2000	2125	1.06	3000	2438	0.81	R. T.
PH 48-4 b	781	2000	2500	1.25	3000	2969	0.99	D. T.
PH 48-4 c	1125	2000	2188	1.09	3000	2938	0.98	F., D. T.
DV 48-5 a	2969	3000	4250+[1]	1.42+	3750	4250+[1]	1.13+	None
DV 48-5 b	2094	3000	3531+[2]	1.08+	3750	3531	0.94	D. T.
PH 48-5 a	1250	3000	2969+[2]	0.99+	3750	2969	0.77	D. T.
PH 48-5 b	1563	3000	3344	1.11	3750	3531	0.94	D. T., R. T.
PH 48-5 c	1344	3000	3469	1.16	3750	3625	0.97	D. T., R. T.
PH 60-2 a	750	1000	1450	1.45	1500	2100	1.40	F.
PH 60-2 b	750	1000	1300	1.30	1500	2150	1.43	F.
PH 60-2 c	950	1000	1275	1.28	1500	1950	1.30	F.
PH 60-3 a	800	1350	1750	1.30	2000	2500	1.25	F., R. T.
PH 60-3 b	800	1350	1650	1.22	2000	2700	1.35	F., D. T.
PH 60-3 c	750	1350	1600	1.19	2000	2750	1.38	F., R. T.
PH 60-4 a	1250	2000	2325	1.16	3000	3200	1.07	D. T., R. T.
PH 60-4 b	1250	2000	2500	1.25	3000	3325	1.11	D. T., R. T.
PH 60-4 c	1300	2000	2600	1.30	3000	3250	1.08	D. T., R. T.
DV 72-2 a	833	1000	1556	1.56	1500	2056	1.37	F., D. T.
DV 72-2 b	667	1000	1222	1.22	1500	1667	1.11	F.
PH 72-2 a	792	1000	1417	1.42	1500	1958	1.31	F.
PH 72-2 b	750	1000	1292	1.29	1500	1667	1.11	F., R. T.
PH 72-2 c	792	1000	1354	1.35	1500	1854	1.24	F.
DV 72-3 a	778	1350	1667	1.23	2000	2000	1.00	Comb.
DV 72-3 b	861	1350	2028	1.50	2000	2611	1.31	D. T.
PH 72-3 a	1042	1350	1583	1.17	2000	2438	1.22	F., R. T.
PH 72-3 b	958	1350	1458	1.08	2000	2104	1.05	F., R. T.
PH 72-3 c	1000	1350	1625	1.20	2000	2458	1.23	D. T.
DV 72-4 a	1222	2000	2778	1.39	3000	2778	0.93	R. T.
DV 72-4 b	1500	2000	3083	1.54	3000	3222	1.07	D. T.
PH 72-4 a	-	2000	3000+[2]	1.50+	3000	3000	1.00	R. T.
PH 72-4 b	1208	2000	2500	1.25	3000	2833	0.94	R. T.
PH 72-4 c	1167	2000	2083	1.04	3000	2500	0.83	R. T.
DV 96-2 a	583	1000	1417	1.42	1500	1667	1.11	D. T., R. T.
DV 96-2 b	583	1000	1333	1.33	1500	1896	1.26	D. T., R. T.
DV 96-3 a	750	1350	1625	1.20	2000	1875	0.94	D. T., R. T.
DV 96-3 b	563	1350	1333	0.99	2000	1833	0.92	D. T., R. T.
DV 96-4 a[3]	708	2000	2792	1.40	3000	3417	1.14	F., R. T.
DV 96-4 b[3]	1333	2000	2708	1.35	3000	3500	1.17	D. T., R. T.
SMOOTH WIRE FABRIC REINFORCEMENT								
DV 48-2P a	875	870[5]	1281	1.47	1300[5]	2250	1.73	F., D. T.
DV 48-2P b	719	870	1422	1.64	1300	2281	1.75	F.
PH 48-2P a	844	870	1469	1.69	1300	2313	1.78	F.
PH 48-2P b	844	870	1375	1.58	1300	2156	1.66	F., R. T.
PH 48-2P c	938	870	1188	1.36	1300	2125	1.64	F.
DV 48-3P a	875	1150[5]	1250	1.09	1750[5]	2531	1.45	F., R. T.
DV 48-3P b	1000	1150	1531	1.34	1750	2656	1.52	F., D. T.
PH 48-3P a	750	1150	1125	0.98	1750	2000	1.14	D. T.
PH 48-3P b	906	1150	1250	1.09	1750	2438	1.39	D. T.
PH 48-3P c	938	1150	1375	1.20	1750	2094	1.20	R. T.
DV 48-4P a	1406	2000	2156	1.08	3000	2625	0.88	D. T.
DV 48-4P b	1250	2000	2594	1.30	3000	3594	1.20	F., D. T.
PH 48-4P a	1031	2000	1875	0.94	3000	2688	0.90	F., R. T.
PH 48-4P b	1125	2000	1875	0.94	3000	2563	0.85	D. T.
PH 48-4P c	1031	2000	1750	0.88	3000	2438	0.81	F., R. T.

DEFORMED WIRE FABRIC

SMOOTH WIRE FABRIC

NOTES

1 Capacity of testing machine reached at before 0.01 inch crack or ultimate D-Load
2 Sample failed before 0.01 inch crack D-Load was reached
3 Pipe contained stirrups
4 For deformed wire series, required D-Load is that specified in C76 for strength class and it is the D-Load for which the pipe was designed using the Methods in Ref. 2. Design is based on nominal dimensions, areas, and concrete strength. Steel areas contain no allowance for manufacturing variability.
5 Areas for smooth wire were made identical to those for deformed wire as described in note 4. This required adjustment in Required D-Load from C76 values to account for lower ultimate strength and lower 0.01 inch crack strength offered by the smooth wire.

KEY

F. = Flexural Failure
D. T. = Diagonal Tension Failure
R. T. = Radial Tension Failure
F, D. T. = Indicates diagonal tension observed at one end and flexural cracks only at other. Probable failure mode is D. T.
Comb. = Combined Flexural and Diagonal Tension Failure

TABLE A4

PIPE DEFLECTIONS DURING TEST

DEFORMED WIRE FABRIC

Pipe Mark	AT 0.01 INCH CRACK LOAD D-Load	Deflection (in)† Vert.	Deflection (in)† Horiz.	AT 1.4 x ASTM C76 - 0.01 INCH CRACK LOAD D-Load	Deflection (in)† Vert.	Deflection (in)† Horiz.
DV 48-2 a	1281	-3/16	1/8	1400	-	-
DV 48-2 b	1688	-3/16	1/16	1400	-	-
PH 48-2 a	1344	-1/16	1/16	1400	-	-
PH 48-2 b	1719	-1/16	1/8	1400	-	-
PH 48-2 c	1438	-3/16	1/8	1400	-	-
DV 48-3 a	1500	-1/8	1/16	1890	-1/8	3/16
DV 48-3 b	1500	-1/16	1/16	1890	-5/16	1/8
PH 48-3 a	1656	-1/8	1/8	1890	-1/4	3/16
PH 48-3 b	1500	-	-	1890	-	-
PH 48-3 c	1375	-1/8	3/16	1890	-1/4	3/8
DV 48-4 a	1813	0	1/8	2800	-5/16	5/16
DV 48-4 b	-	-	-	2800	-	-
PH 48-4 a	2125	-1/8	1/8	2800	-	-
PH 48-4 b	2500	-	-	2800	-	-
PH 48-4 c	2180	-1/4	1/16	2800	-3/8	3/16
DV 48-5 a	-	-3/16	3/16	4200	-	-
DV 48-5 b	-	-	-	4200	-	-
PH 48-5 a	-	-	-	4200	-	-
PH 48-5 b	3344	-1/4	3/16	4200	-	-
PH 48-5 c	3469	-3/16	3/16	4200	-	-
PH 60-2 a	1450	-3/16	1/16	1400	-	-
PH 60-2 b	1300	0	1/8	1400	-5/16	3/16
PH 60-2 c	1275	-3/16	3/16	1400	-1/4	7/32
PH 60-3 a	1750	-1/8	3/16	1890	-7/16	1/4
PH 60-3 b	1650	-3/8	1/16	1890	-9/16	3/16
PH 60-3 c	1600	-1/4	1/8	1890	-3/8	1/4
PH 60-4 a	2325	-1/4	1/4	2800	-	-
PH 60-4 b	2500	-5/16	1/8	2800	-	-
PH 60-4 c	2600	-3/16	3/16	2800	-	-
DV 72-2 a	1556	-1/8	1/4	1400	-	-
DV 72-2 b	1222	-1/4	1/8	1400	-1/4	3/16
PH 72-2 a	1417	-1/16	3/16	1400	-	-
PH 72-2 b	1292	-3/16	1/16	1400	-5/16	1/16
PH 72-2 c	1354	-3/16	1/8	1400	-	-

Pipe Mark	AT 0.01 INCH CRACK LOAD D-Load	Deflection (in)† Vert.	Deflection (in)† Horiz.	AT 1.4 x ASTM C76 - 0.01 INCH CRACK LOAD D-Load	Deflection (in)† Vert.	Deflection (in)† Horiz.
DEFORMED WIRE FABRIC						
DV 72-3 a	1667	-1/4	1/4	1890	-1/2	3/8
DV 72-3 b	2028	-3/8	1/4	1890	-	-
PH 72-3 a	1583	-1/8	3/16	1890	-3/16	1/4
PH 72-3 b	1458	-3/16	3/16	1890	-5/16	5/16
PH 72-3 c	1625	-1/8	3/16	1890	-	-
DV 72-4 a	2778	-1/4	1/2	2800	-	-
DV 72-4 b	3083	-1/2	5/16	2800	-	-
PH 72-4 a	-	-	-	2800	-	-
PH 72-4 b	2500	-3/16	1/4	2800	-1/4	3/8
PH 72-4 c	2083	-	-	2800	-	-
DV 96-2 a	1417	-1/4	3/16	1400	-1/2	7/16
DV 96-2 b	1333	-3/8	5/16	1400	-1	7/8
DV 96-3 a	1625	-1/2	1/4	1890	-3/4	9/16
DV 96-3 b	1333	-5/16	1/8	1890	-	-
DV 96-4 a	2792	-1/4	1/4	2800	-5/16	11/16
DV 96-4 b	2708	-1/4	3/16	2800	-	-
SMOOTH WIRE FABRIC						
DV 48-2P a	1281	-1/8	1/8	1400	-	-
DV 48-2P b	1422	-1/8	1/16	1400	-	-
PH 48-2P a	1469	-3/16	0	1400	-	-
PH 48-2P b	1375	-1/8	1/16	1400	-	-
PH 48-2P c	1188	-	-	1400	-	-
DV 48-3P a	1250	-1/16	1/8	1890	-1/4	1/4
DV 48-3P b	1531	0	1/8	1890	-1/8	1/4
PH 48-3P a	1125	-1/16	1/16	1890	-7/16	7/16
PH 48-3P b	1250	-	-	1890	-	-
PH 48-3P c	1375	-	-	1890	-	-
DV 48-4P a	2156	-3/16	1/8	2800	-	-
DV 48-4P b	2594	-3/16	1/8	2800	-3/16	1/4
PH 48-4P a	1875	-1/4	3/16	2800	-	-
PH 48-4P b	1875	-1/16	1/8	2800	-	-
PH 48-4P c	1750	-3/16	1/8	2800	-	-

NOTES

† Deflection measured is change in diameter, + indicates increase; − indicates decrease
Vert. indicates vertical diameter during test
Horiz. indicates horizontal diameter during test

TABLE A5

COMPARISON OF TEST AND CALCULATED 0.01 INCH CRACK STRENGTHS

PIPE REINFORCED WITH WELDED DEFORMED WIRE FABRIC

Pipe Mark	$DL_{.01\ test}$	$DL_{.01\ calc.}$	$\frac{DL_{.01\ test}}{DL_{.01\ calc.}}$
Packerhead Process Pipe			
PH 48-2 a	1344	1278	1.05
PH 48-2 b	1719	1206	1.43
PH 48-2 c	1438	1323	1.09
PH 48-3 a	1656	1463	1.13
PH 48-3 b	1500	1384	1.08
PH 48-3 c	1375	1437	0.96
PH 48-4 a	2125	2598	0.82
PH 48-4 b	2500	2573	0.97
PH 48-4 c	2188	2332	0.94
PH 48-5 a	2969+	3783	0.79+
PH 48-5 b	3344	4813	0.69
PH 48-5 c	3469	5013	0.69
PH 60-2 a	1450	1071	1.35
PH 60-2 b	1300	966	1.35
PH 60-2 c	1275	1084	1.18
PH 60-3 a	1750	1443	1.21
PH 60-3 b	1650	1286	1.28
PH 60-3 c	1600	1437	1.11
PH 60-4 a	2325	2698	0.86
PH 60-4 b	2500	2629	0.95
PH 60-4 c	2600	2832	0.92
PH 72-2 a	1417	1098	1.29
PH 72-2 b	1292	1026	1.26
PH 72-2 c	1354	1073	1.26
PH 72-3 a	1583	1448	1.09
PH 72-3 b	1458	1495	0.98
PH 72-3 c	1625	1690	0.96
PH 72-4 a	3000+	3054	0.98
PH 72-4 b	2500	2943	0.85
PH 72-4 c	2083	2784	0.75

Pipe Mark	$DL_{.01\ test}$	$DL_{.01\ calc.}$	$\frac{DL_{.01\ test}}{DL_{.01\ calc.}}$
Dry-Pack-Vibration Process Pipe			
DV 48-2 a	1281	1275	1.00
DV 48-2 b	1688	1495	1.13
DV 48-3 a	1500	1672	0.90
DV 48-3 b	1500	1591	0.94
DV 48-4 a	1813	3001	0.60
DV 48-4 b	3219+	2958	1.08+
DV 48-5 a	4250+	4802	0.89+
DV 48-5 b	3531+	3890	0.91+
DV 72-2 a	1556	1500	1.04
DV 72-2 b	1222	1363	0.90
DV 72-3 a	1667	1544	1.08
DV 72-3 b	2028	1798	1.13
DV 72-4 a	2778	3216	0.86
DV 72-4 b	3083	3159	0.98
DV 96-2 a	1417	1265	1.12
DV 96-2 b	1333	1332	1.00
DV 96-3 a	1625	1630	1.00
DV 96-3 b	1333	1777	0.75
DV 96-4 a	2792	2218	1.26
DV 96-4 b	2708	2223	1.22

PIPE REINFORCED WITH WELDED SMOOTH WIRE FABRIC

Pipe Mark	$DL_{.01\ test}$	$DL_{.01\ calc.}$	$\frac{DL_{.01\ test}}{DL_{.01\ calc.}}$
PH 48-2P a	1469	1179	1.25
PH 48-2P b	1375	1200	1.15
PH 48-2P c	1188	1138	1.04
PH 48-3P a	1125	1209	0.93
PH 48-3P b	1250	1408	0.89
PH 48-3P c	1375	1382	0.99
PH 48-4P a	1875	2474	0.76
PH 48-4P b	1875	2193	0.85
PH 48-4P c	1750	2273	0.77

Pipe Mark	$DL_{.01\ test}$	$DL_{.01\ calc.}$	$\frac{DL_{.01\ test}}{DL_{.01\ calc.}}$
DV 48-2P a	1281	1240	1.03
DV 48-2P b	1422	1132	1.26
DV 48-3P a	1250	1395	0.90
DV 48-3P b	1531	1348	1.14
DV 48-4P a	2156	2557	0.84
DV 48-4P b	2594	2696	0.96

NOTE:

Specimens with loads and ratios followed by + failed in diagonal tension or reached capacity of testing machine before a 0.01 inch crack width was obtained. Thus, the 0.01 inch crack strengths of these specimens were higher than the indicated values.

TABLE A6

STATISTICAL PARAMETERS FOR VARIOUS GROUPS OF TEST PIPE–0.01 INCH CRACK STRENGTH

Test Program	Type of Pipe	Manufacturing Process	Group Characteristics		Number of Specimens	Avg. $\frac{DL_{.01\ test}}{DL_{.01\ calc.}}$	Coefficient of Variation %
Present	Pipe with DWWF[1] Reinf.	Packerhead	Entire Group of Pipe		30	1.04+[3]	19.
			48" pipe excluded		18	1.09+[4]	17.
			By Size	48" diameter	12	0.97+[4]	21.
				60" diameter	9	1.13	15.
				72" diameter	9	1.05+[4]	17.
			By Class	II	9	1.25	9.
				III	9	1.09	10.
				IV	9	0.89+[4]	8.
				V	3	0.72+[4]	6.
		Dry-Pack-Vibration	Entire Group of Pipe		20	0.99+[5]	15.
			48" pipe excluded		12	1.03	14.
			By Size	48" diameter	8	0.93+[5]	16.
				72" diameter	6	1.00	10.
				96" diameter	6	1.06	16.
			By Class	II	6	1.03	8.
				III	6	0.97	13.
				IV	6	1.00+[4]	20.
				V[3]	2	0.90+[3]	1.
	Pipe with SWWF[2] Reinf.	Packerhead	Entire Group of Pipe		9	0.96	16.
			By Class	II	3	1.15	9.
				III	3	0.94	4.
				IV	3	0.79	5.
		Dry-Pack-Vibration	All pipe		6	1.02	14.
			By Class	II	2	1.15	10.
				III	2	1.02	12.
				IV	2	0.90	7.
Previous U. S. Steel Ref. 2	Pipe with DWWF Reing.	Cast	All pipe - USS		47	1.03	10.5
			All pipe - Others		20	1.06	15.6
M. I. T. U. S. Steel & ACPA Ref. 4	Pipe with SWWF Reinf.	Cast	All pipe		33	1.03	15.1

1. DWWF = Deformed Welded Wire Fabric
2. SWWF = Smooth Welded Wire Fabric, 8" spacing of longitudinals
3. Two test specimens failed in D.T. before 0.01" crack appeared. True0.01" crack strength would increase ratio.
4. One test specimen failed in D.T. before 0.01" crack reached. True0.01" crack strength would increase ratio.
5. Three test specimens failed in D.T. before 0.01" crack appeared. True 0.01" crack strength would increase ratio.

TABLE A7

COMPARISON OF TEST AND CALCULATED ULTIMATE STRENGTH

PIPE REINFORCED WITH WELDED DEFORMED WIRE FABRIC

Pipe Mark	DL_u Test	Mode of Failure Test	DL_u calc. for Flex. Failure	DL_u calc. for D. T. Failure	$\frac{DL_u\ test}{DL_u\ calc.}$ Flex. Failure	$\frac{DL_u\ test}{DL_u\ calc.}$ D. T. Failure
Packerhead Process Pipe						
PH 48-2 a	2281	F., D. T.	2298	2399	0.99	0.95
PH 48-2 b	2406	F.	2404	2571	1.00	0.94
PH 48-2 c	2219	F.	2414	-	0.92	-
PH 48-3 a	2625	F., R. T.	2652	2587	0.99	1.01
PH 48-3 b	2563	F.	2424	2272	1.06	1.13
PH 48-3 c	2500	F., D. T.	2418	2397	1.03	1.04
PH 48-4 a	2438	R. T.	-	3137	-	0.78
PH 48-4 b	2969	D. T.	-	3103	-	0.96
PH 48-4 c	2938	F., D. T.	-	2876	-	1.02
PH 48-5 a	2969	D. T.	-	3790	-	0.78
PH 48-5 b	3531	D. T., R. T.	-	4466	-	0.79
PH 48-5 c	3625	D. T., R. T.	-	4618	-	0.78
PH 60-2 a	2100	F.	1634	-	1.28	-
PH 60-2 b	2150	F.	1530	-	1.41	-
PH 60-2 c	1950	F.	1670	1809	1.17	1.08
PH 60-3 a	2500	F., R. T.	2237	2164	1.12	1.16
PH 60-3 b	2700	F., D. T.	2065	1967	1.31	1.37
PH 60-3 c	2750	F., R. T.	2021	2098	1.36	1.31
PH 60-4 a	3200	D. T., R. T.	-	2969	-	1.08
PH 60-4 b	3325	D. T., R. T.	-	2914	-	1.14
PH 60-4 c	3250	D. T., R. T.	-	3163	-	1.03
PH 72-2 a	958	F.	1549	-	1.26	-
PH 72-2 b	1667	F., R. T.	1453	-	1.15	-
PH 72-2 c	1854	F.	1467	-	1.26	-
PH 72-3 a	2438	F., R. T.	2140	2043	1.14	1.19
PH 72-3 b	2104	F., R. T.	2181	2020	0.96	1.04
PH 72-3 c	2458	D. T.	2297	2321	1.07	1.06
PH 72-4 a	3000	R. T.	-	3213	-	0.93
PH 72-4 b	2833	F., R. T.	-	3152	-	0.90
PH 72-4 c	2500	F., R. T.	-	2986	-	0.84

Pipe Mark	DL_u Test	Mode of Failure Test	DL_u calc. for Flex. Failure	DL_u calc. for D. T. Failure	$\frac{DL_u\ test}{DL_u\ calc.}$ Flex. Failure	$\frac{DL_u\ test}{DL_u\ calc.}$ D. T. Failure
Dry-Pack-Vibration Process Pipe						
DV 48-2 a	2281	F.	2453	2516	0.93	0.91
DV 48-2 b	2438	F, R. T.	2631	2677	0.93	0.91
DV 48-3 a	2906	F., D. T.	2749	2860	1.06	1.02
DV 48-3 b	2813	F.	2703	2807	1.04	1.00
DV 48-4 a	3000	D. T.	-	3506	-	0.86
DV 48-4 b	3219	D. T.	-	3376	-	0.95
DV 48-5 a	4250+	None	-	4536	-	0.94+
DV 48-5 b	3531	D. T.	-	3927	-	0.90
DV 72-2 a	2056	F., D. T.	1865	-	1.10	-
DV 72-2 b	1667	F.	1787	-	0.93	-
DV 72-3 a	2000	Comb.	2483	2241	0.81	0.89
DV 72-3 b	2611	D. T.	2797	2544	0.93	1.03
DV 72-4 a	2778	-	-	3325	-	0.84
DV 72-4 b	3222	D. T.	-	3202	-	1.01
DV 96-2 a	1667	D. T., R. T.	1824	1880	0.91	0.89
DV 96-2 b	1896	D. T., R. T.	1831	1990	1.04	0.95
DV 96-3 a	1875	D. T., R. T.	-	2155	-	0.87
DV 96-3 b	1833	D. T., R. T.	-	2217	-	0.83
DV 96-4 a	3417	F., R. T.	-	2536	-	1.35*
DV 96-4 b	3500	D. T., R. T.	-	2505	-	1.40*

PIPE REINFORCED WITH WELDED SMOOTH WIRE FABRIC

Pipe Mark	DL_u Test	Mode of Failure Test	DL_u calc. for Flex. Failure	DL_u calc. for D. T. Failure	$\frac{DL_u\ test}{DL_u\ calc.}$ Flex. Failure	$\frac{DL_u\ test}{DL_u\ calc.}$ D. T. Failure
PH 48-2P a	2313	F.	2435	2679	0.95	0.86
PH 48-2P b	2156	F., R. T.	2477	-	0.87	-
PH 48-2P c	2125	F.	2275	2528	0.93	0.84
PH 48-3P a	2000	D. T.	2315	2134	0.86	0.94
PH 48-3P b	2438	D. T.	2569	2548	0.95	0.96
PH 48-3P c	2094	R. T.	2425	2520	0.86	0.83
PH 48-4P a	2688	F., R. T.	-	3361	-	0.80
PH 48-4P b	2563	D. T.	-	2926	-	0.88
PH 48-4P c	2438	F., R. T.	-	3052	-	0.80

Pipe Mark	DL_u Test	Mode of Failure Test	DL_u calc. for Flex. Failure	DL_u calc. for D. T. Failure	$\frac{DL_u\ test}{DL_u\ calc.}$ Flex. Failure	$\frac{DL_u\ test}{DL_u\ calc.}$ D. T. Failure
DV 48-2P a	2250	F., D. T.	2409	2606	0.93	0.86
DV 48-2P b	2281	F.	2208	2400	1.03	0.95
DV 48-3P a	2531	F., R. T.	2557	2762	0.99	0.92
DV 48-3P b	2656	F., D. T.	2474	2663	1.07	1.00
DV 48-4P a	2625	D. T.	-	3566	-	0.74
DV 48-4P b	3594	F., D. T.	-	3784	-	0.95

* Specimens contained stirrups

NOTE: Specimens with loads and ratios followed by + reached capacity of testing machine without failure and thus have strengths higher than the indicated values.

TABLE A8

STATISTICAL PARAMETERS FOR VARIOUS GROUPS OF TEST PIPE—ULTIMATE STRENGTH

Test Program	Type of Pipe	Manufacturing Process	Group Characteristics	Flexural Failure: Number of Specimens	Flexural Failure: Avg. $\frac{DL_{u\,test}}{DL_{u\,calc.}}$	Flexural Failure: Coefficient of Variation %	Diagonal Tension Failure: Number of Specimens	Diagonal Tension Failure: Avg. $\frac{DL_{u\,test}}{DL_{u\,calc.}}$	Diagonal Tension Failure: Coefficient of Variation %
Present	Pipe with DWWF[1] Reinf.	Packerhead	Entire Group of Pipe	11	1.17	13.	19	1.00	16.
			48" pipe excluded	8	1.25	8.	10	1.07	14.
			By Size 48" diameter	3	0.97	4.	9	0.92	13.
			By Size 60" diameter	4	1.30	7.	5	1.15	10.
			By Size 72" diameter	4	1.19	7.	5	0.98	13.
			By Class II	9	1.16	13.	0	-	-
			By Class III	2	1.22	12.	7	1.14	10.
			By Class IV	0	-	-	9	0.96	11.
			By Class V	0	-	-	3	0.79	0.
		Dry-Pack-Vibration	Entire Group of Pipe	8	0.99	7.	10	0.93+[3]	7.
			48" pipe excluded	4	1.00	8.	6	0.91	8.
			By Size 48" diameter	4	0.99	6.	3	0.91+[3]	4.
			By Size 72" diameter	2	1.02	8.	4	0.94	8.
			By Size 96" diameter	2	0.97	6.	2	0.85	3.
			By Class II	6	0.97	7.	0	-	-
			By Class III	2	1.05	1.	4	0.90	8.
			By Class IV	0	-	-	4	0.89	11.
			By Class V	0	-	-	2	0.92+[3]	2.
	Pipe with SWWF[2] Reinf.	Packerhead	Entire Group of Pipe	4	0.90	4.	5	0.87	8.
			By Class II	3	0.92	4.	0	-	-
			By Class III	1	0.86	0.	2	0.95	11.
			By Class IV	0	-	-	3	0.83	5.
		Dry-Pack-Vibration	Entire Group of Pipe	4	1.01	5.	2	0.84	13.
			By Class II	2	0.98	5.	0	-	-
			By Class III	2	1.03	4.	0	-	-
			By Class IV	0	-	-	2	0.84	13.
Previous U.S. Steel	Pipe with DWWF Reinf.	Cast	All pipe	9	0.98	6.5	57	1.01	11.8

1. DWWF = Deformed Welded Wire Fabric
2. SWWF = Smooth Welded Wire Fabric, 8" maximum spacing of longitudinals
3. Failure load not reached in one 48 inch diameter test specimen

Appendix B

NOTATION

The following notation is used in this paper:

A_{s1} = steel area of inside cage, square inches per linear foot of pipe wall;

A_{s2} = steel area of outside cage, square inches per linear foot of pipe wall;

d_1 = depth of section from compressive edge of concrete to center of inside tensile reinforcement, inches;

d_2 = depth of section from compressive edge of concrete to center of outside tensile reinforcement, inches;

D_i = inside diameter of pipe, inches;

DL_u = ultimate D-load capacity of pipe, pounds per linear foot of length per foot of diameter;

$DL_{0.01}$ = 0.01-in. crack D-load capacity of pipe, pounds per linear foot of length per foot of diameter;

ϕ_f = variability factor for design based on ultimate flexural strength requirements;

$\phi_{0.01}$ = variability factor for design based on 0.01-in. crack strength requirements;

ϕ_d = variability factor for design based on ultimate diagonal tension strength requirements; and

ϕ_x = variability factor for variations in materials and fabrication.