

Research on Bolt Failures in Wolf Creek Structural Plate Pipe

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An investigation of high strength bolts in $\frac{3}{8}$ -in. thick structural plate was undertaken following discovery of an 18 $\frac{1}{2}$ -ft diameter pipe failure at Wolf Creek, Montana, in July 1964. Fractured A 490 bolts were found to meet ASTM hardness and chemistry requirements. Static load seam strength tests on $\frac{3}{4}$ -in. diameter A 325 and A 490 bolts in $\frac{3}{8}$ -in. corrugated structural plate were made to determine effects of initial bolt torque, reduced temperatures, freezing water, and prying action from plate bending. No evidence of bolt or joint weakness was observed.

Further research and tests showed that A 490 bolts comparable to Wolf Creek bolts (R_C 38) were susceptible to hydrogen embrittlement. Environmental conditions required for cathodic charging of atomic hydrogen were present in the Wolf Creek pipe. Sustained bolt stress levels above a threshold value apparently led to hydrogen-stress cracking. Hydrogen-stress cracking offers a logical explanation of the field failure.

•IN July 1964, joint failure of an 18 $\frac{1}{2}$ -ft diameter structural plate pipe on the Wolf Creek Canyon Interstate Highway in Montana was discovered. Several hundred bolts had broken in the structural plate seams allowing the plates to telescope circumferentially 4 to 5 ft in places.

This culvert structure is 548 ft long under a maximum rock fill height of 83 ft. In the center portion, where fill loads are greatest, $\frac{3}{8}$ -in. thick plates are used with $\frac{3}{4}$ -in. ASTM A 490¹ high strength bolts. Near the ends lighter gage plates are used with ASTM A 325 bolts (Fig. 1).

The structure was erected and backfilled in November 1963. Construction of the embankment was temporarily halted in January 1964, due to winter weather conditions. Until then, alignment of the structure was checked using plumb bobs inside the pipe. Any failure up to that time would certainly have been noticed. Construction was resumed in the latter part of March and completed in April 1964. The failure was not discovered until early in July. It is possible that the structure was not observed for the 6-month period between January and July.

Standard practice is the use of $\frac{3}{4}$ -in. A 325 high strength bolts in "Multi-Plate" structures. For $\frac{3}{8}$ -in. thick plate, however, $\frac{3}{4}$ -in. A 490 bolts were used to provide an increase in the bolt shear capacity. Installation torques were stipulated to be 200 ft-lb minimum and 400 ft-lb maximum². It is the opinion of several engineers who inspected the failed pipe that the stipulated maximum torque was exceeded. The degree of

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¹Formerly referred to by ASTM designation A 354 Grade BD; for $\frac{3}{4}$ -in. diameter bolts the significant properties are identical.

²Application of high strength bolts in structural plate joints is not to be confused with use of high strength bolts in usual structural bearing or friction-type connections; such connections usually consist of lapped flat plate elements connected by means of bolts tightened to proof load initial tension.

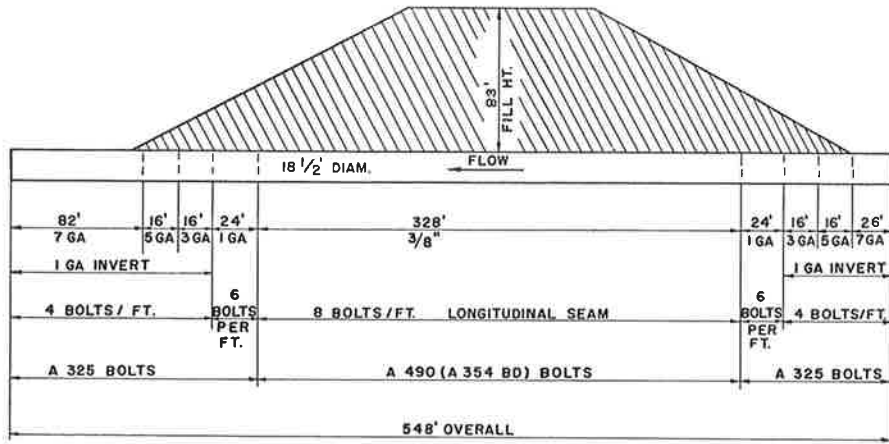


Figure 1. Wolf Creek structural plate pipe.

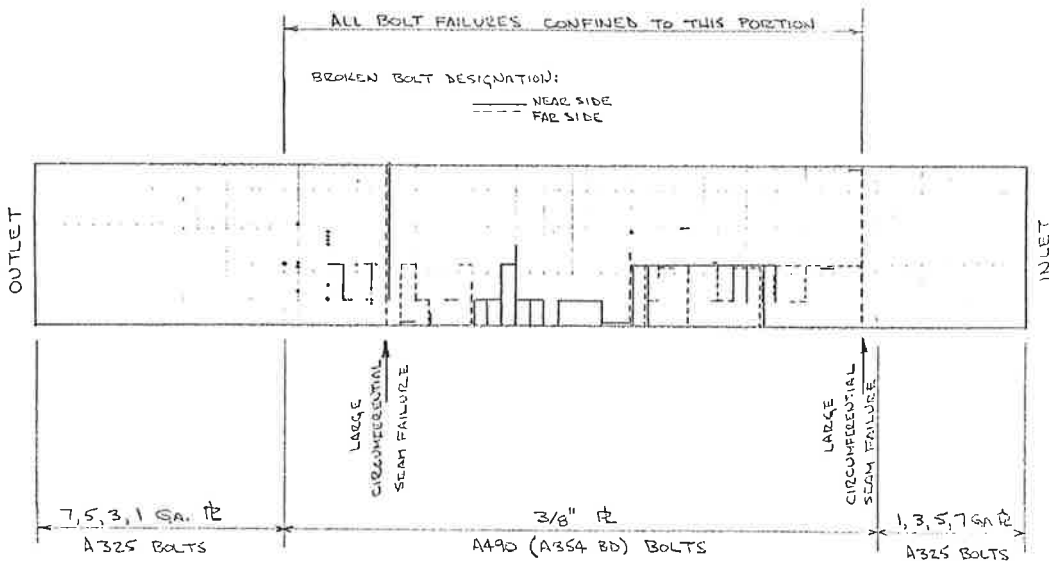


Figure 2. Diagram of failure.

nut disfiguration and the apparent "dimpled" effect at the bolt holes require torque in excess of 400 ft-lb.

All failed bolts were A 490 in $\frac{3}{8}$ -in. plate. No failures occurred among the A 325 bolts used in the lighter gage plates. Beginning at the first ring of $\frac{3}{8}$ -in. plate from the outlet end of the pipe, a few bolts were broken in the seams in the lower half of the pipe. No movement of the plates had occurred up to 194 ft from the outlet end, at which point there was a large circumferential seam failure. Another large circumferential seam separation occurred 90 ft from the inlet end of the pipe, which was one ring removed from the juncture of $\frac{3}{8}$ -in. and 1-gage rings. Between these 2 circumferential failures, the plates in the lower half of the pipe had telescoped to within 1 to 2 ft of each other at longitudinal seams, reducing the pipe diameter to approximately 16 to 17 ft. These seams were boltless with the nut or head of bolt missing. Several other longitudinal and circumferential seams had failed bolts in this portion of the structure.

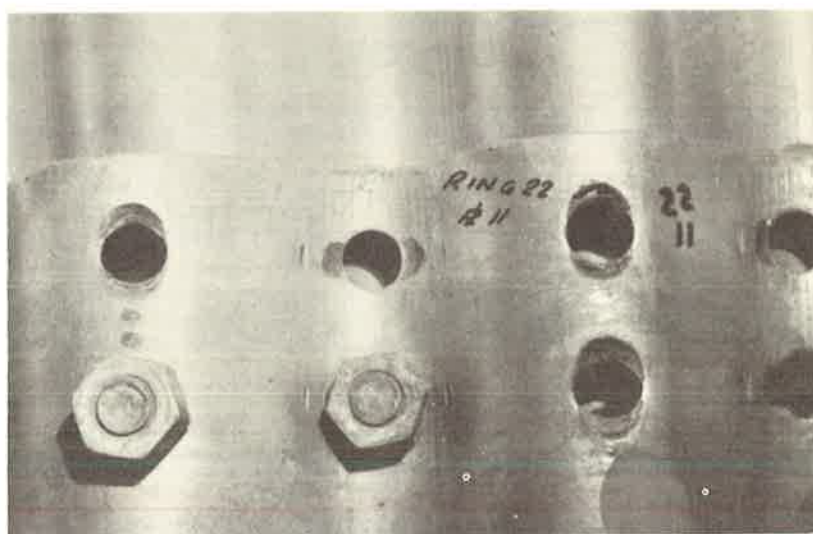
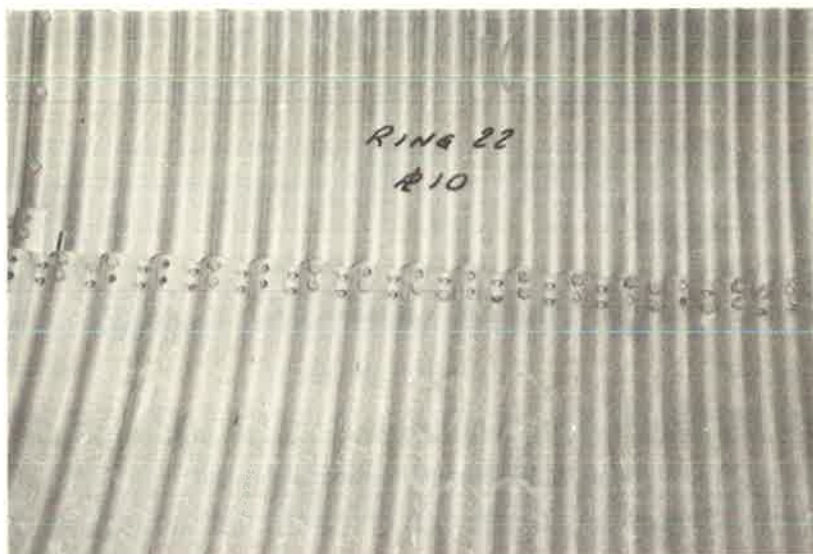


Figure 3. Longitudinal seam—no deformation of plate material at bolt holes.

Figure 2 is a diagram of the failure showing the location of broken bolts and the pattern of failure.

Engineers inspecting the failure reported that in most instances (except where circumferential seam bolts had obviously sheared to accommodate the change in shape of the failed structure) there was no evidence of any distortion of the plate at the bolt holes (Fig. 3).

After the failure had been discovered, it was quite difficult to assess the primary cause by visual observation of the structure. The failure was sufficiently catastrophic so that several types of failure were evident.

TABLE 1
BOLT PROPERTIES

Hardness (Determined in accordance with ASTM A 370 Sec F4)					Chemistry (Check analysis)						Tensile Strength (Avg of 3 tests on full size bolts)			
Bolts	Rockwell C Hardness		ASTM Specs.		Elements	Bolts Used in Investigation					Bolts Used in Investigation	Tensile Strength (lb)	ASTM Spec.	
	Min	Max	Min	Max		Wolf Creek	$\frac{3}{4}$ in. A 490	ASTM A 490 Spec.	$\frac{3}{4}$ in. A 325	ASTM A 325 Spec.				
1	37.0	37.5	32	38	Carbon	0.48	0.47	0.28 to 0.55	0.34	0.27 min	$\frac{3}{4}$ in. A 490	57,400	50, 100 min	
2	36.5	37.5			Manganese	0.86	0.93		0.82	0.47 min	$\frac{3}{4}$ in. A 325	45,600	40, 100 min	
3	38	38.5			Phosphorus	0.020	0.013	0.045 max	0.013	0.048 max				
4	36.5	37			Sulfur	0.026	0.024	0.045 max	0.021	0.058 max				
5	36.5	38.5			Nickel	0.19	0.15							
6	38				Molybdenum	0.19	0.13							
Bolts used in investigation					Chromium	0.93	0.91							
$\frac{3}{4}$ in. A 325	30	30.5	23	35										
$\frac{3}{4}$ in. A 490	34	38	32	38										



Figure 4. Broken bolts obtained from Wolf Creek.

BOLT AND PLATE PROPERTIES

Approximately 40 broken bolts were obtained from Wolf Creek following discovery of the failure. Hardnesses determined on 6 Wolf Creek bolts are given in Table 1. Check analysis chemistry is also given. Hardness and chemistry meet ASTM requirements indicating that failure was not due to defective bolt material.

Generally, the fracture surface of the bolts was of a brittle nature. The fractures occurred in the threads at various distances from the nut. If bolt shear had been the mode of failure, the fracture surface would be along the shear plane, which is one plate thickness from the nut for lapped joints. Several of the fracture surfaces were rather flat and perpendicular to the axis of the bolt. This type of fracture could possibly be identified with torsion-tensile failure (Fig. 4).

For the purpose of investigating the behavior of high strength bolts in $\frac{3}{8}$ -in. thick structural plate, supplies of both A 325 and A 490 galvanized Armco "Multi-Plate" bolts were obtained. These bolts were manufactured in the same manner as those used

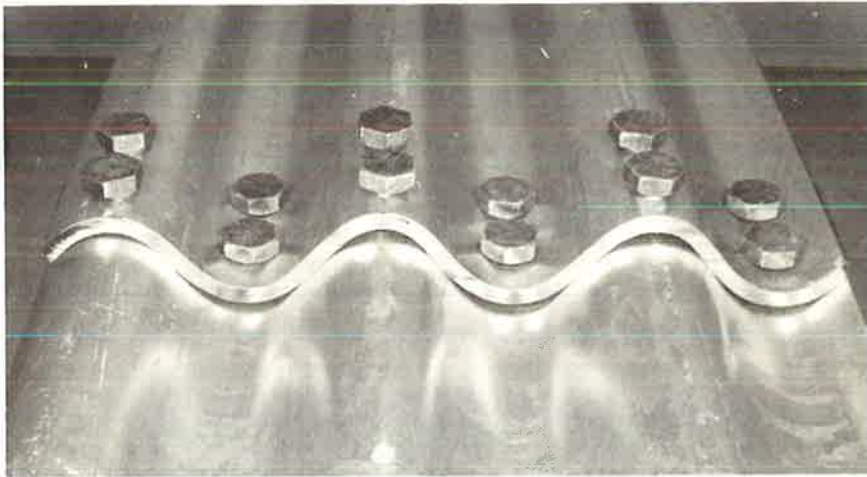


Figure 5. Fit between $\frac{3}{8}$ -in. plates with bolts torqued to 200 ft-lb.

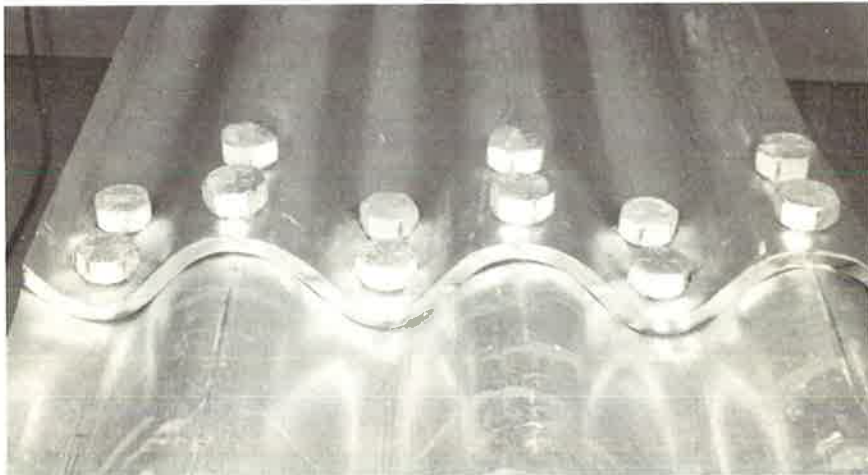


Figure 6. Fit between $\frac{3}{8}$ -in. plates with bolts torqued to 910 ft-lb.

in the Wolf Creek pipe. ASTM A 325 is a quenched and tempered carbon steel bolt, and A 490 is a quenched and tempered alloy steel bolt. Properties of these bolts are also given in Table 1. Note that properties of the A 490 bolts acquired for the investigation are close to the actual Wolf Creek bolts.

Both grades of bolts were inserted in a joint of $\frac{3}{8}$ -in. structural plate and torqued to failure to determine ultimate torque values. Average results from 2 tests using the galvanized nuts and bolts as received were: (a) A 490 ultimate torque = 950 ft-lb, and (b) A 325 ultimate torque = 840 ft-lb.

Structural plate material used in the investigation was standard 6- x 2-in. corrugation, $\frac{3}{8}$ -in. thick, hot-dip galvanized, copper-bearing steel.

When structural plate sections are connected by the single lap joint, the corrugations do not nest in intimate contact. Inherent gaps between plates at crests and valleys of the corrugations will depend on the plate thickness. The greater the thickness, the greater the gap distance. Figure 5 illustrates the gaps present in $\frac{3}{8}$ -in. thick plate

with the bolts tightened to 200 ft-lb torque. A torque of approximately 800 ft-lb was required to essentially close the gaps in 6-in. wide joints. In a wide plate joint complete yielding of the cross section would be required to close the gaps. Figure 6 shows the fit between plates with A 490 bolts tightened to 910 ft-lb.

Any efforts in the field to draw the $\frac{3}{8}$ -in. plates tightly together would require an excessive value of torque resulting in high initial bolt stress.

SEAM STRENGTH TESTS

It has been stated that the bolt fractures appeared to be brittle. Temperatures inside the pipe could have conceivably fallen to the transition temperature of the bolt material. By this hypothesis, the high initial bolt stresses together with the notch effect from the threads might then result in a static brittle fracture of the bolts.

To investigate this possibility, bolted joints were assembled using both A 325 and A 490 bolts torqued to 89 percent of ultimate torque. These joints were lowered to -75 F in a dry ice bath. In one instance the A 490 assembly was subjected to approximately -320 F using liquid air. However, no bolt failures occurred either statically or by impact from a hammer blow.

It was then considered desirable to determine the effect of reduced temperatures on seam strength capacity. Another important factor related to the Wolf Creek failure needing study was the effect of high torques. Consequently, joint strength tests were made using A 325 and A 490 bolts with both initial bolt torque and temperature as variables.

Figures 7 and 8 show the type of specimen and test used to determine compressive seam strength. Testing facilities limited specimen width to 6 in. Four bolts were used corresponding to an 8-bolt-per-foot longitudinal seam. Three levels of torque and three levels of temperature were used in this experiment. Bolts were torqued to: (a) 200 ft-lb, which would be adequate for field installations; (b) torque corresponding to the tensile yield strength of the bolts; and (c) torque of at least 95 percent of the ultimate torque (Table 2).

Yield strength torques were determined by a calibration method using bolts with machined ends. These bolts were tightened to various levels of torque and measured before and after torque release. From these measurements of bolt length it was possible to establish the approximate point of nonlinear response, or permanent set, which was then taken as the yield strength torque.

All A 490 bolted joint compression specimens failed by plastic buckling and yielding of the plate material away from the bolted lap (Fig. 9). In some tests at reduced temperatures the "Multi-Plate" material cracked under high local stresses between the bolt holes (Fig. 10). A 325 bolted specimens failed in either of two modes—plate failure (Fig. 10) or bolt shear (Fig. 11).

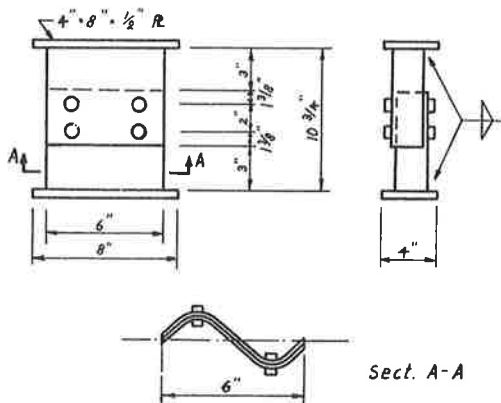


Figure 7. Joint strength compression test specimens.

No bolt shear failures were obtained in the laboratory tests on A 490 bolts. However, in the Wolf Creek pipe all broken bolts were A 490 bolts and none of the A 325 bolts failed. Several characteristics of the A 325 laboratory shear rupture surfaces worth noting are: (a) flowed metal across the bolt ruptures (Fig. 11) was plainly evident; (b) the rupture surface was consistently 10 to 15 deg from the perpendicular cross section of the bolts; and (c) threads of the bolt indented in the plate under bearing pressure. None of these effects were noticeable in the Wolf Creek failure, indicating that bolt shear was not the primary cause of the field failure.

Results of this experiment showed that the effect of initial tightening torque and

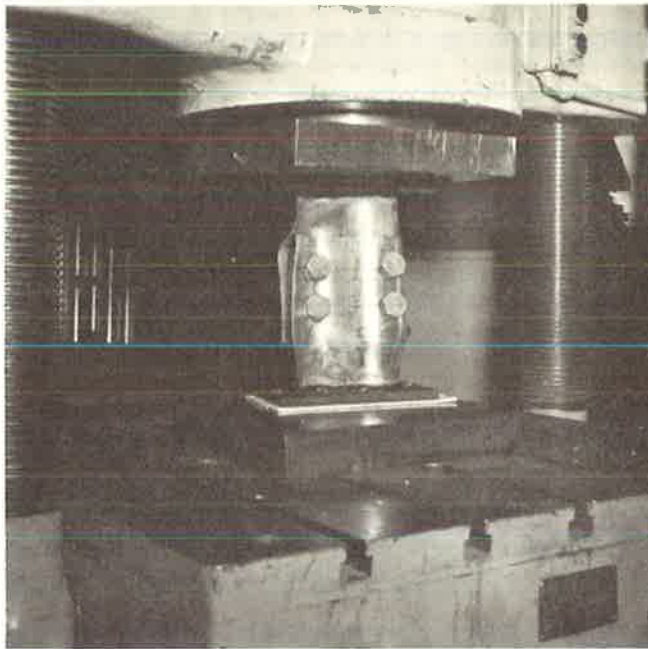


Figure 8. Joint strength test set-up.

TABLE 3
RESULTS OF JOINT STRENGTH TESTS

A 325 Bolts ^a				A 490 Bolts ^a			
Temp. (°F)	Torque, Ft-Lb			Temp. (°F)	Torque, Ft-Lb		
	200	630 (75%)	825 (98%)		200	710 (75%)	910 (96%)
Room Temp.	122,200 lb—bolt shear, some plate failure	128,900 lb—bolt shear, some plate failure	121,500 lb—bolt shear, slight plate failure	Room Temp.	120,000 lb—plate failure	129,000 lb—plate failure	130,600 lb—plate failure
-10	126,800 lb—plate failure, crack in plate	129,800 lb—plate failure, crack in plate	Retorqued ^c to 825 ft-lb: 116,900 lb—same failures as above	-10	127,600 lb—plate failure	131,800 lb—plate failure, crack in plate	Retorqued ^c to 910 ft-lb: 133,200 lb—same failure as above
-60	127,200 lb—plate failure, crack in plate	128,500 lb—bolt shear, crack in plate, plate failure	122,200 lb—bolt shear, crack in plate, slight plate failure	-60	129,800 lb—plate failure, crack in plate	130,000 lb—plate failure, crack in plate	135,500 lb—plate failure, crack in plate

^a 3/4-in. bolts, 6-in. wide specimens with 4 bolts.^b Torque values shown in parentheses are percent of ultimate torque.^c Tests were run at room temperature after bolts retorqued to maximum value. Loss in torque of 80-86 percent.

specimen temperature on seam strength was of questionable significance. Wallaert and Fisher (1) have recently reported that initial bolt preload from tightening did not significantly affect the shear strength of A 490 bolts.

Bending moment applied to a lapped connection will cause tensile forces in the fasteners due to prying action, the magnitude depending on the rigidity of the connected parts. Structural plate in 3/8-in. thickness might conceivably provide sufficient rigidity to "pry" the bolts to tensile failure.

This possible explanation of the Wolf Creek failure was investigated by making beam tests on bolted structural plate joints. Forty-nine specimens, 1 in. long by 18 in. wide, were prepared with the bolted joint located at mid-span. The following 4 tests were made:



Figure 9. Typical plate failure.

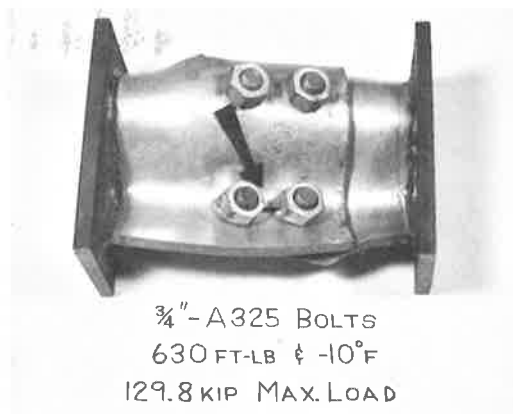


Figure 10. Typical plate failure with crack in plate between bolt holes in lower row.

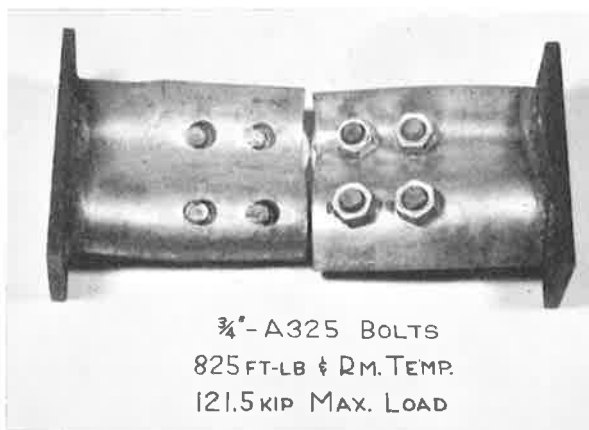


Figure 11. Typical bolt shear failure of A 325 bolts.

- $\frac{3}{4}$ -in. - A 490 bolts at 200 ft-lb,
- $\frac{3}{4}$ -in. - A 490 bolts at 910 ft-lb,
- $\frac{3}{4}$ -in. - A 325 bolts at 200 ft-lb, and
- $\frac{3}{4}$ -in. - A 325 bolts at 825 ft-lb.

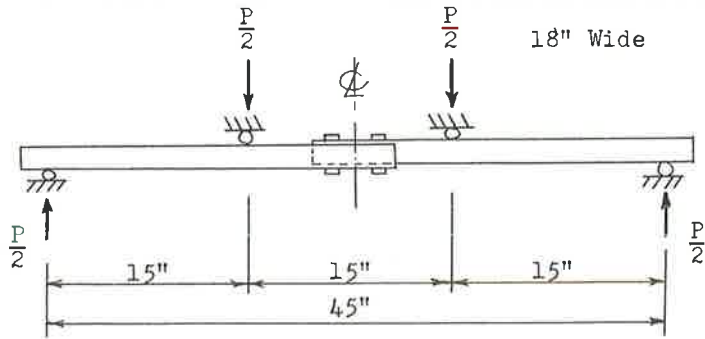
Third-point loads were applied with an overall span of 45 in. (Fig. 12 and Table 3).

Ultimate loads were governed by plastic bending of the structural plate along a cross section at a row of bolts without any bolt tensile failures. The bending strength or rigidity of the $\frac{3}{8}$ -in. plate was lower than that required to cause prying forces equal to the tensile strength of the bolts. A typical beam test specimen after testing is shown in Figure 13.

An opinion was made that water freezing inside the lapped joints might create pressure sufficient to cause tensile failure of the bolts. It is true that most of the failed bolts were near or below water. To check this hypothesis, joint specimens were prepared such that water could be held between the plates. The freezing action did not fail the bolts.

The data show that the A 490 bolts were not mechanically defective as manufactured, and that the bolt failures could not be explained on any basis of mechanical weaknesses.

TABLE 3
BEAM TEST RESULTS



Test No.	Bolts	Torque, ft-lb	Maximum Load, P (lb)	Equiv. Moment, $\frac{\text{in.} \cdot \text{kip}}{\text{ft}}$	Failure Mode
1	A 490	200	26,250	131	Plate plastic bending
2	A 490	910	30,100	150	
3	A 325	200	28,100	141	
4	A 325	825	30,000	150	

Note: Avg maximum moment = 143 in.-kip per ft of plate

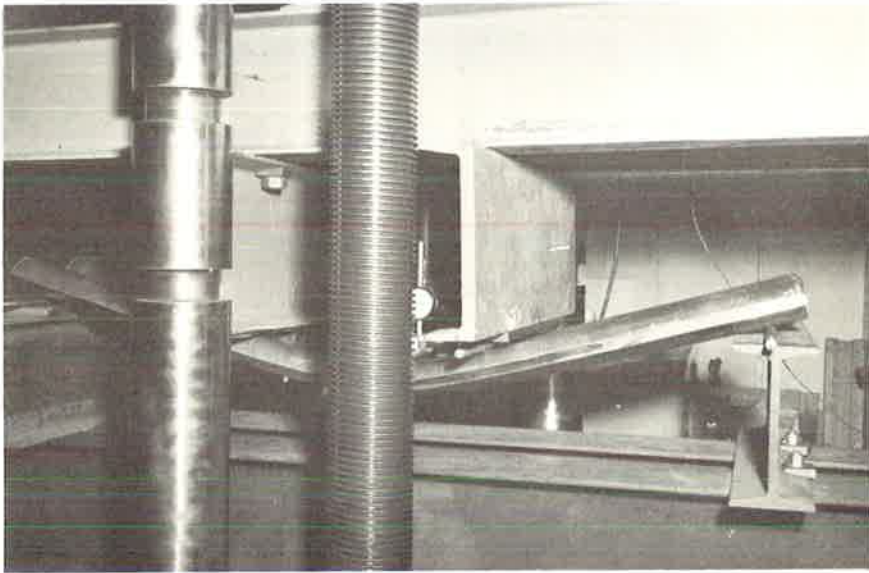


Figure 12. Beam test set-up. Bolted splice is behind screw column.

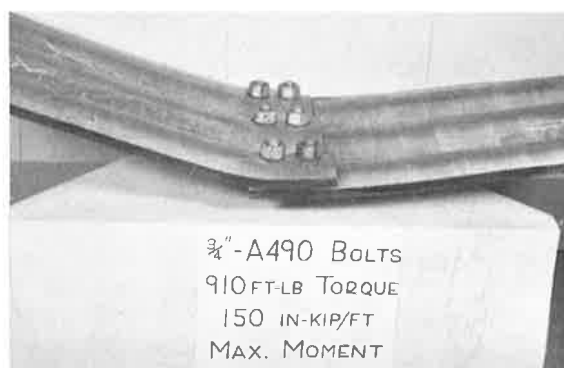


Figure 13. Typical beam test specimen after testing showing plastic bending failure of plate material.

Because the failures (Fig. 2) were near or below the water level, it appeared that an environmental effect such as corrosion might be the primary cause of failure.

ASPECTS OF HYDROGEN EMBRITTLEMENT

If a steel is sufficiently hard and if it absorbs more than some minimum amount of atomic hydrogen, a sustained tensile stress may, in time, cause a crack leading to fracture. Moreover, the applied stress may be much lower than that required to cause fracture in the absence of hydrogen. Such failures are known to occur with very little ductility even though in a tension test the material exhibits normal ductility (2). These delayed, brittle failures can occur in all types of high strength steels except possibly those of strictly austenitic microstructure, although even this exception remains a matter of conjecture.

Although hydrogen embrittlement and hydrogen-stress cracking have been known and studied for many years (3), the mechanisms are still debatable. Actually, there is not yet a precise distinction between hydrogen embrittlement and stress-corrosion cracking in high strength steels. All of the theories advanced to explain the effects depend on a critical combination of hydrogen and stress (4).

Atomic hydrogen can be absorbed by the steel from cleaning (pickling) operations, plating operations, cathodic protection (5), or corrosion processes (6, 7). Extremely small amounts of hydrogen in conjunction with sufficient stress levels have been shown capable of causing delayed fracture (7). Under most conditions, the strength or hardness level of high strength steel under tensile stress is one important material factor affecting hydrogen embrittlement susceptibility. Alloy composition is a relatively unimportant factor (4). The susceptibility diminishes with decreasing strength or hardness and also diminishes with decreasing applied stress. For a given hardness level, time-to-fracture curves become asymptotic at an applied stress referred to as a threshold stress. No clearly delineated threshold stress has been established (3). However, numerous investigators have verified the concept that the minimum or threshold stress for hydrogen-stress cracking decreases for increasing tensile strength or hardness levels.

In many electrochemical cells hydrogen evolves at the cathode. Holidays or coating defects in high strength steel parts coated with zinc, cadmium, aluminum, etc., in the presence of an electrolyte are enough to set up an electrochemical cell. The exposed steel becomes the cathode. Atomic hydrogen so produced may readily enter the steel, and if it is under sufficient sustained tensile stress, delayed hydrogen embrittlement fracture may occur. Impregnating the steel with atomic hydrogen is sometimes referred to as "cathodic charging."

Conditions necessary for cathodic charging were present in the Wolf Creek structure. The structural plates and bolts were hot-dip galvanized. Prior to galvanizing, the bolts

TABLE 4
CHEMICAL ANALYSIS OF LITTLE PRICKLY CREEK
WATER DETERMINED BY MONTANA STATE
BOARD OF HEALTH^a

Property	Content (ppm)
Total hardness (as CaCO ₃)	200
pH	8.2
CO ₂	trace
Dissolved oxygen (at 45 F)	11.4
H ₂ S	none
Total dissolved solids	200
Chloride (Cl)	10
Sulphate radical (SO ₄)	14
Nitrates (NO ₃)	none
Carbonate radical (CO ₃)	3
Bicarbonate radical (HCO ₃)	217
Total alkalinity (CaCO ₃)	183
(1) Methyl Orange test	5
(2) Phenolphthalein test	178
Calcium (Ca)	40
Magnesium (Mg)	24
Iron (Fe)	0.16
Sodium and potassium calc. (Na-K)	8
Fluoride (F)	0.7

^a Little Prickly Creek is the stream that runs through the pipe, Wolf Creek is the name of the town nearby, and Lyons Creek South refers to the highway project. Sample taken 11/4/65, analysis reported 11/5/65.

were pickled in the standard practice of manufacture. It has been stated that pickling is a potential source of hydrogen embrittlement. However, this explanation does not appear to be valid in this case because of the pattern of failures being below probable water levels.

It is reasonable to assume that the zinc coating on the bolt threads was damaged during installation enough to expose some bare steel. Water flowing in and permeating the soil around the culvert could serve as the electrolyte. The stream water analysis is given in Table 4. This analysis undoubtedly varies throughout the year depending on the amount of rainfall and melting snow.

Exploratory tests were made to determine the hydrogen-stress cracking susceptibility of the A 325 bolts at R_C 30 hardness and the A 490 bolts at R_C 38 hardness. Bolts were loaded as short cantilever beams in bending by using the fixture shown in Figure 14. A small magnesium disk was used as the anode. This was done in an attempt to produce failures—if they would occur—in a reasonable length of time. It took 2 to 8 months in the field. This was too long for a lab test. The high hardness (R_C 38) bolt assembly shown in Figure 14 was placed in a 3 to 4 percent salt solution and cracked as seen overnight. In a similar test a softer (R_C 30) bolt did not crack after 92 hr.

Loading a bolt in tensile and torsional shear stresses by tightening the nut provides a better representation of actual application. To this end bolts were tightened in a sleeve-type block with windows in its sides for observing cracks (Fig. 15). Magnesium bars were connected to the block and then the complete assembly was placed in tap water. A high hardness (R_C 38) bolt tightened to approximately 80 percent of the

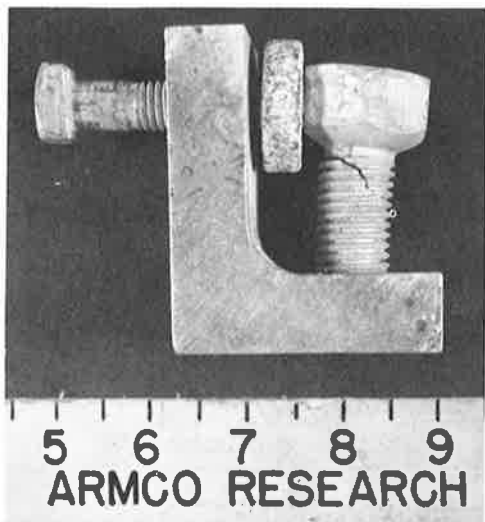


Figure 14. Preliminary test fixture used to determine hydrogen embrittlement cracking susceptibility of "Multi-Plate" bolts.

ultimate torque cracked after 4 days (Fig. 15). A softer bolt (R_C 30) tested in the same manner had not cracked after 70 days. Numerous other tests on the low and high hardness bolts were made with attempts to vary the magnitude of stress, the electrolyte, and the method of charging. The same trend of results was obtained; i. e., the high hardness A 490 bolts (approximately R_C 38) showed much greater susceptibility to hydrogen-stress cracking than the softer A 325 bolts (approximately R_C 30).

These exploratory tests suggested that bolts at a hardness level comparable to the Wolf Creek bolts could be susceptible to hydrogen embrittlement. No evidences of bolt or joint weaknesses were observed in the static strength tests previously described. Therefore, a hydrogen-stress cracking hypothesis is offered as an explanation for the field failure. Field conditions tending to substantiate this hypothesis are:

1. The failure was of a delayed type since the bolts fractured sometime between January and July.
2. Failed bolts were predominately near and below the assumed water level in the pipe where a corrosion process could have occurred (Fig. 2).
3. Only the A 490 bolts failed. These bolts had the maximum hardness of the permissible range— R_C 38. No failures occurred in the softer A 325 bolts.

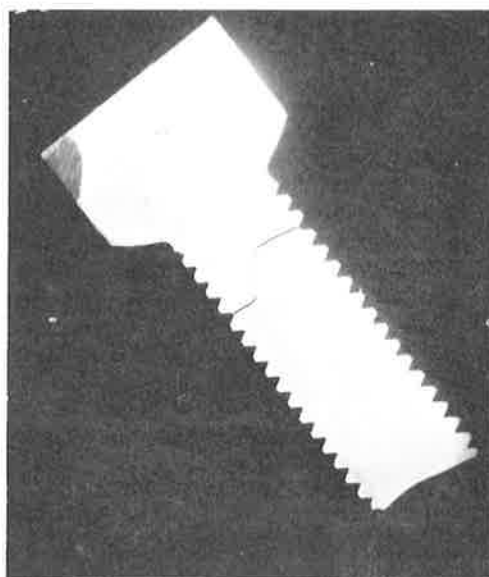
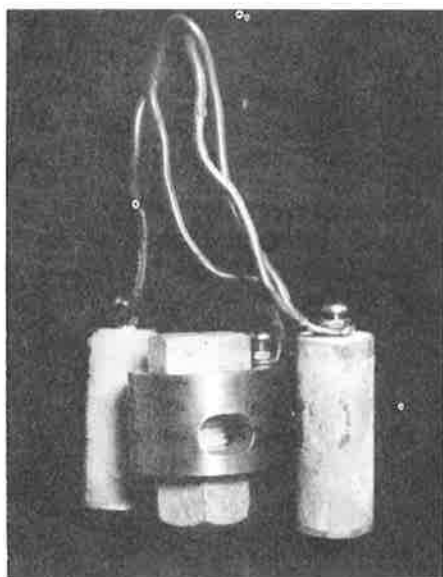


Figure 15. Bolt stressed by tightening in "solid" block: at right, cracks in R_C 38 (A 490) bolt after 4 days in tap water.

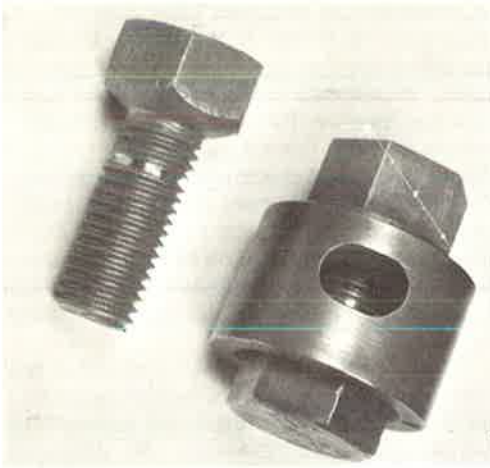


Figure 16. Machined bolt specimen and test block assembly for hydrogen-stress cracking experiments.

machined and polished to a depth slightly below the normal root diameter. This groove controlled the location of the hydrogen induced crack (Fig. 16). Bolts were tightened in the test block to desired tensile stresses by using the calibrated torque-tension relationship discussed later. The assembly was coated with wax leaving an exposed portion of the groove for hydrogen entry at each window opening in the test block. Specimens were placed in a 5 percent sulfuric acid solution containing approximately 0.05 percent sodium arsenite. A direct current of 1.5 to 2.5 amperes was maintained between the specimen and a Duriron bar anode (Fig. 17). At 15-min intervals the specimens were removed and examined under magnification for cracks (Table 5).

Time-to-fracture or first crack is used as a measure of hydrogen embrittlement susceptibility. Figure 18 shows time to crack and applied stress for both hardness levels. These data are in reasonable agreement with other investigations (5).

The threshold stress for R_C 38 bolts was found to be approximately 20,000 psi, while the value for R_C 25 bolts was approximately 48,000 psi. Actual cathodic charging

4. Tensile stresses were highest in the A 490 bolts as they were used exclusively in the $\frac{3}{8}$ -in. plates where greater torques were required to pull the plates together. A 325 bolts were used only in the lighter gage—1, 3, 5, and 7 plates.

5. The structure and bolts were galvanized.

6. Fractured bolts generally appeared to be brittle tensile failures.

7. There were instances of a few broken bolts in longitudinal seams which had not been displaced. These failures indicated that something besides shear weakness led to fracture.

Following the preliminary tests, hydrogen-stress cracking experiments were made under more controlled conditions, with different levels of sustained tensile stress on A 490 bolts. To determine the effect of hardness, a group of A 490 bolts was heat-treated to approximately R_C 25. An annular groove was

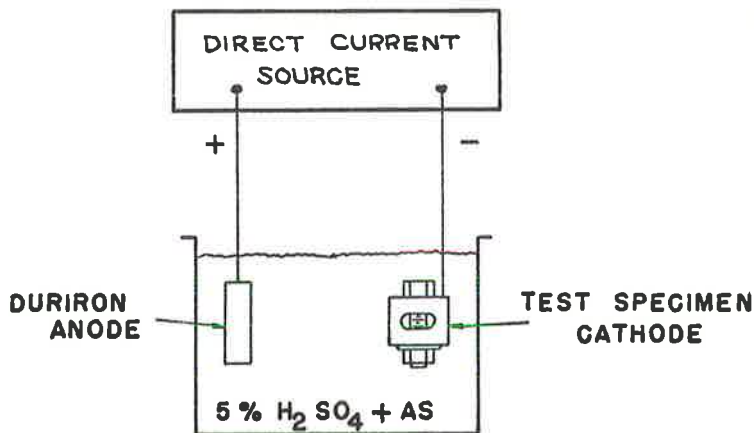
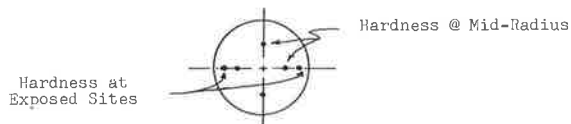


Figure 17. Hydrogen test method.

TABLE 5
RESULTS OF HYDROGEN-STRESS CRACKING TESTS UNDER
CONTROLLED TENSION



Bolt No.	Applied Torque, ft-lb	Average Tensile Stress, psi	Hardness at Exposed Sites, R _C	Hardness at Mid-Radius, R _C		Time to First Crack
				Min	Max	
25-5	115	48,000	26-27	27	28.5	no cracks (23 hr)
25-6	115	48,000	25-24.5	24	24.5	no cracks (24 hr)
25-7	115	48,000	25-27	22.5	26.5	no cracks (29 hr)
25-2	170	68,000	24-24	24	25	210 min
25-3	170	68,000	27.5-24	26.5	27.5	75 min
25-4	170	68,000	25.5-25	24.5	26.5	120 min
25-8	170	68,000	29.5-20.5	28	30	105 min
38-11	40	20,000	38-38	38	38.5	no cracks (23 hr)
38-16	40	20,000	37-40	38.5	39	no cracks (24 hr)
38-17	40	20,000	39.5-40	39	39	no cracks (24 hr)
38-12	80	34,000	38.5-39	37	39.5	30 min
38-4	115	48,000	38-37	37	38	90 min
38-9	115	48,000	38.5-39	38	39	15 min
38-10	115	48,000	39.5-39.5	39	40	45 min
38-14	115	48,000	39-40	39	40.5	15 min
38-15	115	48,000	39.5-39.5	39	40.5	15 min
38-3	170	68,000	37.5-39	38	38.5	30 min
38-6	170	68,000	39-38.5	39	39	30 min
38-13	170	68,000	39.5-40	39	39.5	15 min
38-19	170	68,000	38-39	37.5	38	15 min
38-2	250	97,000	38-38	37	38.5	15 min
38-5	250	97,000	39-38.5	39	40	15 min
38-7	250	97,000	39-39	39.5	40.5	45 min
38-18	250	97,000	39-40	39	40	15 min

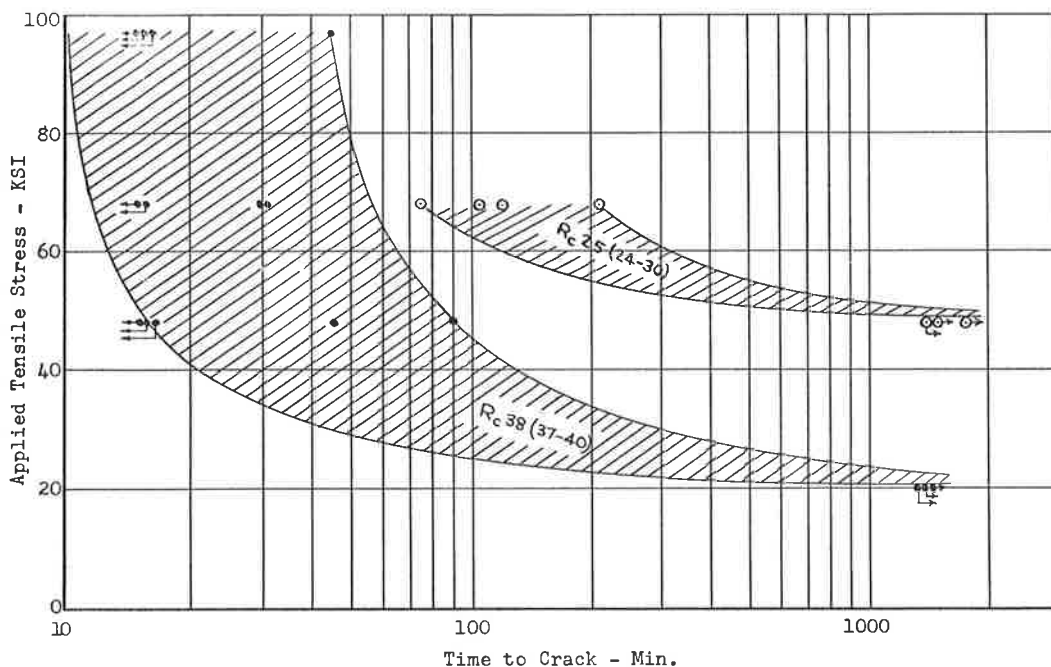


Figure 18. Hydrogen-stress cracking experiments.

conditions expected in the field would, of course, be much less severe. The time factor associated with hydrogen-stress cracking in the field would be much greater.

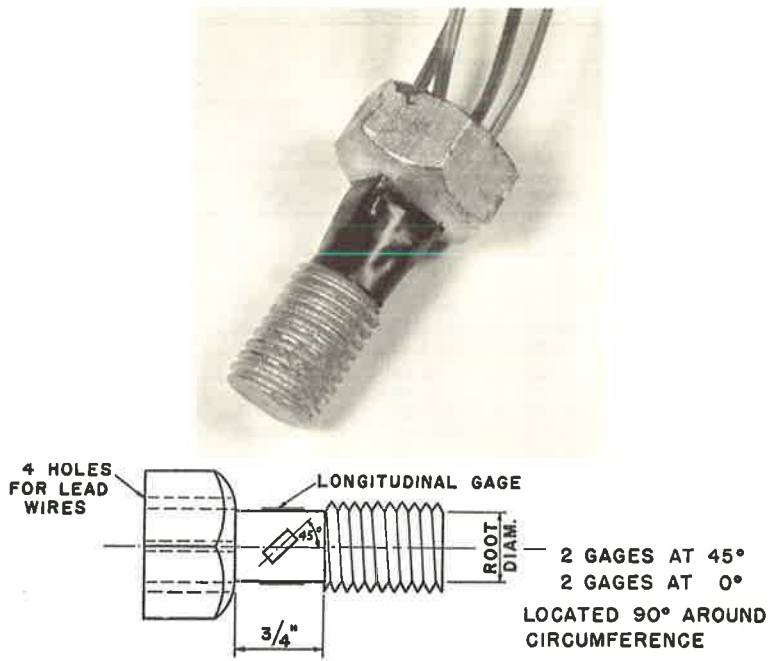


Figure 19. Strain gage instrumented bolts.

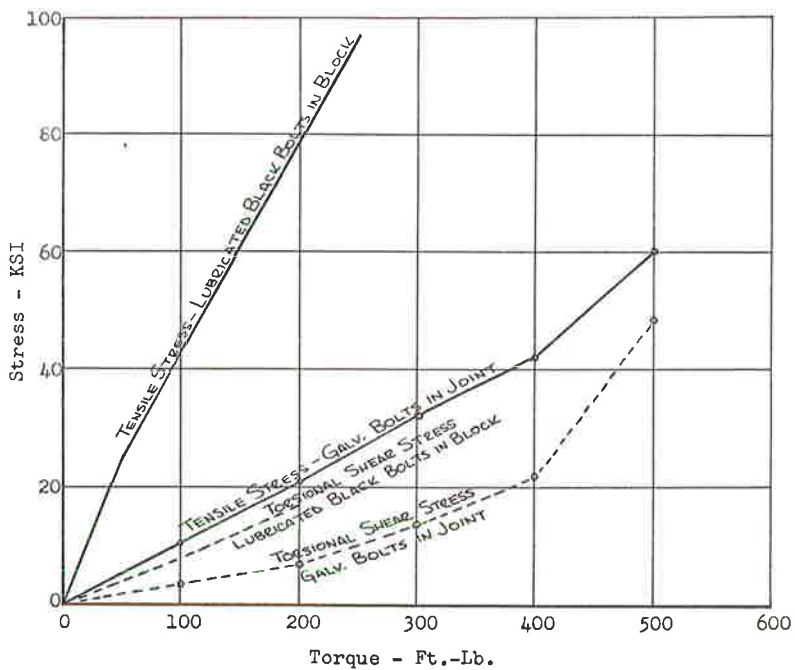


Figure 20. Comparison of bolt stresses vs torque.

BOLT STRESSES RELATED TO APPLIED TORQUE

In usual structural joints using high strength bolts for friction-type or bearing-type connections, the bolts are installed by either the calibrated wrench method or the turn-of-nut method (8). Present-day specifications and practice no longer refer to torque values for installing bolts to minimum bolt tension values³. The commonly used turn-of-nut method involves certain nut rotations from a condition of connected parts being in full contact with each other ("snug tight"). This procedure is therefore not applicable to structural plate joints since the corrugations are inherently un-nestable. The concepts of bolting heavy structural joints are not applicable to structural plate pipe joints.

For this investigation it was desirable to ascertain the tensile and torsional shear stresses in the structural plate bolts as related to applied torque. Stresses were determined from strain gage instrumented bolts (Fig. 19). From this gage arrangement it is possible to determine tensile and torsional stresses for elastic conditions. Formulas for computing tensile stresses and torsional shear stresses are given in the Appendix.

One series of tests was made on instrumented bolts in the galvanized condition tightened in a galvanized structural plate joint. The stress-torque relationship is shown in Figure 20. Note that the torsional shear stress begins to increase significantly for applied torques above approximately 400 ft-lb. For increasing values of torque, torsional shear stress increases due to increasing friction developed by: (a) galling of the zinc at the thread contact surfaces, and (b) wedging action of the spherical seated nut into the hole in the plate.

Similar tests were made to calibrate tensile stress values for the hydrogen-stress cracking experiments. It was necessary to provide for a consistent torque-tension relationship for the hydrogen tests. To accomplish this the zinc was removed from the bolts, special nuts were made and hardened to give a better fit between bolt and nut threads, and the threads were lubricated. Bolts were tightened in the hydrogen test block (Fig. 16). Lubricant was applied to the contact surfaces between nut and block (Fig. 20).

The black bolts with lubricated threads and hardened nuts tightened in the "solid" block are stressed predominately in tension, while the torque applied to galvanized nuts and bolts in a galvanized structural plate joint results in high torsional shear stresses (Fig. 20).

SUMMARY

This investigation of high strength bolts in $\frac{3}{8}$ -in. thick structural plate was undertaken following discovery of the Wolf Creek, Montana, pipe failure in July 1964. The Wolf Creek structure is an 18 $\frac{1}{2}$ -ft diameter pipe culvert installation under an 83-ft high fill. Several hundred A 490 (A 354 BD) bolts were found broken in the longitudinal and circumferential seams in the $\frac{3}{8}$ -in. plate portion of the culvert. Failed bolts were predominately located in the lower half of the pipe. In 40 percent of the culvert structure, near the ends, lighter gage plates were connected with the normally-used A 325 bolts. No bolt failures occurred in these portions of the culvert. Failed bolts obtained from the Wolf Creek pipe were found to meet ASTM Designation A 490 hardness and chemistry requirements.

In an effort to analyze and resolve the failure, seam strength tests were made using both $\frac{3}{4}$ -in. A 325 and A 490 bolts in $\frac{3}{8}$ -in. structural plate joints. It is known that bolts in the field structure were installed in many instances with excessively high values of torque. To check influence of high torques and low temperatures, joint-strength tests were made with bolts torqued to 95 percent of breaking torque and at reduced temperatures to -60 F. These tests showed that high initial torques and/or low temperatures had negligible effect on joint strength.

The laboratory static load test failure mode for $\frac{3}{8}$ -in. plate with $\frac{3}{4}$ -in. A 490 bolts (8-bolt-per-foot seam) is plate buckling and yielding, not bolt shear. Observations of

³In usual structural connections, bolts are tightened to a tension at least equal to the bolt proof load.

the field failures indicated that bolt shear was not the primary mode of failure, although several modes of failure were evident in the final catastrophic state of the culvert pipe.

Since many of the failed bolts appeared to be tensile-type fractures with little ductility, the possibility of tensile failure from prying action during bending was investigated. However, bending moment applied to the joints in beam tests resulted in plastic bending of the structural plate with no bolt tensile failures from prying action.

No evidence of bolt or joint weakness was observed in any of the static strength tests made in the laboratory in search of a plausible explanation of field failure.

Laboratory tests revealed a high degree of susceptibility of A 490 bolts at R_C 38 hardness to brittle fracture when cathodically charged. Cathodic charging of atomic hydrogen could have occurred in the Wolf Creek pipe because (a) the structure was galvanized; (b) installing bolts can result in damaged coating on the threads leaving exposed steel to be cathodic in the electrochemical cell; and (c) water was present to serve as an electrolyte. As a result of over-torqueing, the bolts probably were under sustained stress that was greater than a threshold value necessary for hydrogen-stress cracking.

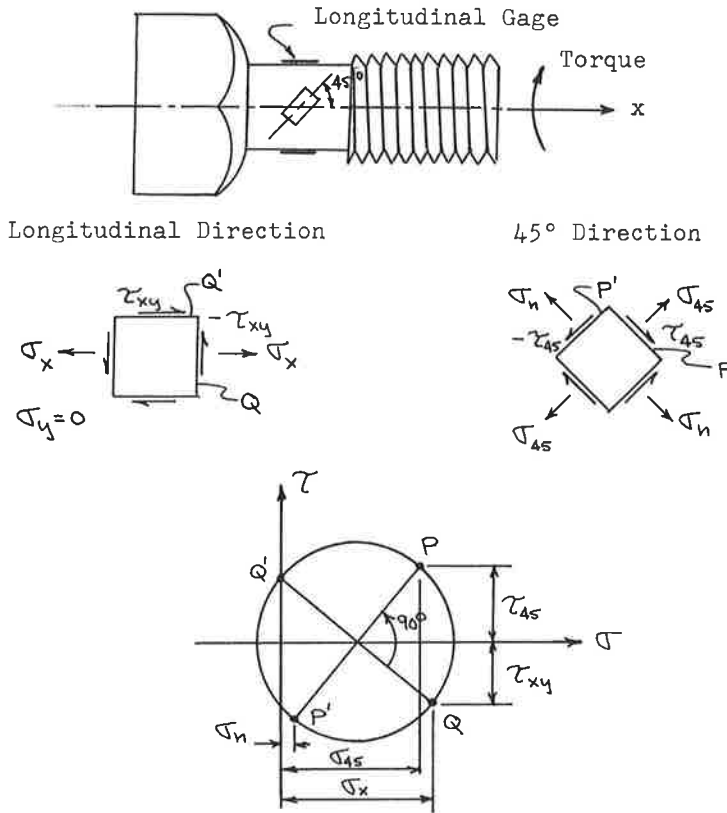
Only high hardness A 490 bolts generally located below water failed. It was a delayed-type failure occurring in a period between 2 and 8 months after erection. The fracture surfaces of broken bolts appeared to be brittle tensile failures. These points provide circumstantial evidence tending to support a corrosion induced hydrogen-stress cracking hypothesis. This hypothesis offers a logical explanation of the field failure. Its mechanism more closely matches the observed field conditions than do mechanisms attributable to simple bolt weakness or to general overload of the structure.

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Appendix

ANALYSIS OF BOLT STRESS FROM STRAIN GAGES



$$\epsilon_x = \frac{\sigma_x}{E} - \frac{\mu \sigma_y}{E}$$

$$\sigma_y = 0$$

$$\epsilon_x = \frac{\sigma_x}{E}$$

$$\sigma_x = \epsilon_x \cdot E$$

ϵ_x from average (to cancel bending strains) of longitudinal gages.

$$\epsilon_{45} = \frac{\sigma_{45}}{E} - \frac{\mu \sigma_n}{E}$$

From geometry of Mohr's circle diagram:

$$\sigma_{45} = \frac{\sigma_x}{2} + \tau_{xy}$$

$$\sigma_n = \frac{\sigma_x}{2} - \tau_{xy}$$

$$\therefore E \cdot \epsilon_{45} = \sigma_{45} - \mu \sigma_n = \frac{\sigma_x}{2} + \tau_{xy} - \mu \left(\frac{\sigma_x}{2} - \tau_{xy} \right)$$

$$E \cdot \epsilon_{45} = \frac{\sigma_x}{2} (1 - \mu) + \tau_{xy} (1 + \mu)$$

$$\text{or } \tau_{xy} = \frac{E \cdot \epsilon_{45} - \frac{\sigma_x}{2} (1 - \mu)}{1 + \mu}$$

ϵ_{45} from average (to cancel bending strains) of 45° gages.

General Discussion

JOHN L. RUMPF, Drexel Institute of Technology, and Chairman, Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation—The Research Council on Riveted and Bolted Structural Joints has conducted research on A 490 bolts at several universities during the past several years. On this basis, the Council has written a specification for the use of these bolts in structural connections.

In March 1965, at the annual meeting of the Council, John Macadam delivered a preliminary report on his research relative to the bolt failures at Wolf Creek. At that time the Council set up a committee to investigate the problem of stress corrosion in high strength bolts. No report is available at this time.

The Council also has research on galvanized bolts under way at 2 universities that may be of interest in connection with this problem.

The above remarks are principally informative. The main point to be made, however, is that the Council feels it would be extremely unfortunate if, because of the Wolf Creek incident, the minds of structural engineers were to be prejudiced against the use of A 490 bolts for structural connections.

M. G. SPANGLER, Research Professor of Civil Engineering, Iowa State University, Ames—These papers constitute an important milestone in the search for a sound and workable procedure for the structural design of underground conduits, particularly highway culverts of the flexible type. The authors and their sponsoring agencies are to be congratulated for making this information available to the engineering profession. Too often, in the past, data on failures of this kind have been stamped "confidential," and the profession at large has not been permitted to make independent studies of the phenomena involved. The result has been that the art and science of design of this type of structure has not progressed as rapidly and as far as it might otherwise have done.

The writer has been associated with the extensive research on underground conduits conducted by the Iowa Engineering Experiment Station over the past 40 yr, and has

found much of interest in studying the Wolf Creek pipe failure in light of this research. The study leads to the conclusion that the cause of failure of the bolted longitudinal seams can be explained by ordinary garden variety structural mechanics. The longitudinal seam bolts were simply over-stressed in shear and direct tension. It is not necessary to reach far out for a somewhat nebulous hypothesis of corrosion and hydrogen embrittlement of the bolts.

In the first place the value of 105 pcf of the embankment soil, which was used in design, probably was too low by about 20 percent. Because the load on the structure is directly proportional to unit weight of the fill material, the design load was likewise too low. The Marston Theory (5) of loads on underground conduits, which is extensively used in this field of endeavor, is very specific with reference to the linear relationship between load and unit weight. Kraft and Eagle state that the lower value of unit weight was used because of ". . . the anticipated arching action in the rock embankment, and accepted practice in reduced unit weight for culvert design" It is this writer's opinion that the practice referred to is accepted only to a very limited extent, and that the best and most usual practice is to use the actual unit weight of the overburden soil in culvert design.

The action of the rock fill over the culvert was probably similar to that which would have occurred if the fill had been soil of the same unit weight as the actual fill. In a study of the structural action of 3 cast iron pipe culverts (2) in northeast Iowa made some 35 years ago, the writer arrived at the following conclusion, among others: "There is every evidence that the rock embankments over these culverts acted in a manner entirely analogous to the action of ordinary earth embankments."

Failure of bolts in the longitudinal seams below the spring line was caused by over-stress in the bolts under a combination of direct shear, due to tangential thrust in the pipe wall, and tensile stress due to bending moment. The pipe was designed by the Ring Compression Theory. A basic weakness of this theory is its failure to recognize the existence of bending moment in the pipe wall. Actually, there is substantial bending moment around the periphery which must be transferred from one ring plate to the next, along with the tangential thrust, by the longitudinal bolted seam connection. This moment exerts a prying action at the seam, which causes a direct tensile stress in the bolts. This direct tension acts simultaneously with shear in the bolts caused by the tangential thrust. The bolts must be properly spaced and strong enough to resist this composite action.

Calculations of stresses in the bolts have been made using the Marston Theory (5) of loads on conduits and the Spangler Theory (3) of the structural action of flexible pipe culverts. Some of the facts required in the application of these theories are not available and it is necessary to deduce or assume values, based on very limited or non-existent information. One of these is the height of fill at which the failure began. The evidence indicates that no one had observed the inside of the structure between the time that the fill was at a height of 10 ft above the pipe, which was in November or December 1963, and the time of discovery of the failure, which was in July 1964. The fill height of 83 ft had been completed early in the spring of 1964.

A second important fact, which is unavailable, is the deflection of the pipe at the time failure began to develop. An estimate of this deflection has been made from the plan view of the pipe after discovery of the failure (Fig. 11 of the Kraft and Eagle paper). Here it is indicated that the horizontal diameter of the pipe had increased by approximately 0.9 ft at the locations of the major circumferential seam failures toward each end of the pipe. These points were under fill heights of 34 and 58 ft, respectively. It has been the writer's observation that the deflection of pipes of this kind is roughly proportional to the height of fill at any point. Therefore, assuming that failure occurred at approximately the final height of 83 ft of fill, the deflection may have been in the neighborhood of 20 in.

For purposes of load and strength calculations, the following assumptions are employed:

1. The vertical load on the pipe is uniformly distributed over the top 180 deg.
2. The bottom reaction is uniformly distributed over the bottom 90 deg of the pipe.

3. Passive lateral pressures act against the sides of the pipe. These are distributed parabolically over the middle 100 deg. The maximum unit pressure acts at the spring line and is equal to the modulus of soil reaction multiplied by one-half the horizontal deflection of the pipe and divided by the radius.

A graphical representation of this load system is shown in Figure 1. Further assumptions are:

$$H = 83 \text{ ft}, B_c = 18.6 \text{ ft}, \frac{H}{B_c} = 4.47$$

$$r = 9.3 \text{ ft} = 111.6 \text{ in.}$$

$$w = 130 \text{ pcf}$$

$$p = 0.9$$

$$r_{sd} = 0, r_{sd}P = 0, C_c = 4.47$$

$$\Delta X = 20 \text{ in.}$$

$$E = 30,000,000 \text{ psi}$$

$$I = 0.226 \text{ in.}^4/\text{in.}, EI = 6,780,000 \text{ in.}$$

$$D_1 = 1.0$$

$$K = 0.096$$

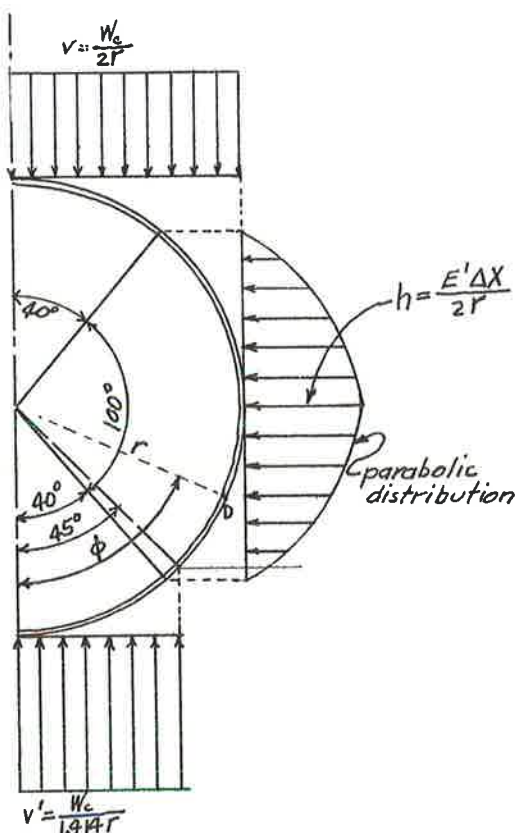


Figure 1. Assumed load system on Wolf Creek culvert.

Calculation of load on the pipe by the Marston formula,

$$W_c = C_c w B_c^2 \quad (1)$$

$$W_c = 4.47 \times 130 \times 18.6^2 = 201,000 \text{ plf}$$

Calculation of modulus of soil reaction by the Spangler formula,

$$\Delta X = D_1 \frac{K W_c r^3}{EI + 0.061 E' r^3} \quad (2)$$

$$E' = 1240 \text{ psi}$$

$$h = \frac{E' \Delta X}{2r} = \frac{1240 \times 20}{2 \times 111.6} = 111 \text{ psi}$$

The units of h are pounds per inch of circumference, per inch length of pipe.

The equations for bending moment, M_D , and tangential thrust, R_D , in the pipe wall are most conveniently written separately for the vertical and horizontal loads (3).

For vertical loads these are:

When ϕ lies between 0 and 45 deg

$$M_D = W_{Cr} (0.183 - 0.26 \cos \phi - 0.354 \sin \phi) \quad (3)$$

$$R_D = W_C (0.026 \cos \phi + 0.707 \sin^2 \phi) \quad (4)$$

When ϕ lies between 45 and 90 deg

$$M_D = W_{Cr} (0.36 - 0.026 \cos \phi - 0.5 \sin \phi) \quad (5)$$

$$R_D = W_C (0.026 \cos \phi + 0.5 \sin \phi) \quad (6)$$

When ϕ lies between 90 and 180 deg

$$M_D = W_{Cr} (0.110 - 0.026 \cos \phi - 0.25 \sin^2 \phi) \quad (7)$$

$$R_D = W_C (0.026 \cos \phi + 0.5 \sin^2 \phi) \quad (8)$$

For horizontal pressure loads, the equations are:

When ϕ lies between 0 and 40 deg

$$M_D = hr^2 (0.345 - 0.511 \cos \phi) \quad (9)$$

$$R_D = hr 0.511 \cos \phi \quad (10)$$

When ϕ lies between 40 and 140 deg

$$M_D = hr^2 (0.199 - 0.5 \cos^2 \phi + 0.143 \cos^4 \phi) \quad (11)$$

$$R_D = hr (\cos^2 \phi - 0.568 \cos^4 \phi) \quad (12)$$

When ϕ lies between 140 and 180 deg

$$M_D = hr^2 (0.345 + 0.511 \cos \phi) \quad (13)$$

$$R_D = hr 0.511 \cos \phi \quad (14)$$

Evaluation and combination of these equations yield the moment diagram shown in Figure 2, and the tangential thrust diagram of Figure 3. The maximum moment in the pipe wall is 63 ft-kips per foot at the bottom. The moment at the longitudinal seam near the lower quarter point ($\phi = 45^\circ$) is approximately 40 ft-kips. A section through the seam is shown in Figure 4. Since the seam must transmit moment from one plate to another, we may determine the tension in the bolts by writing moments about the point of rotation of the seam and setting this equation equal to the moment in the wall.

A difficulty arises in trying to identify the fulcrum or point of rotation, and present knowledge does not permit a rational selection of the actual point. However, it is at least possible that rotation might occur about the inner row of bolts, and since the rows were only 2 in. apart, the tension in the outer row would be $40 \div 0.167 = 240$ kips per ft. Since there are 4 bolts per ft in the outer row, the tensile stress in each bolt is 60 kips. ASTM A 354BD bolts, $\frac{3}{4}$ -in. diameter, have a minimum specified ultimate strength of 50.1 kips. Therefore it is probable that failure of the structure began by tensile failure of the bolts in the lower quarter point seams. Since the calculated stress in the bolts is greater than the minimum specified strength, the factor of safety is less than unity. This suggests that failure probably occurred at some height of fill less than the maximum of 83 ft above the culvert.

This failure hypothesis ignores the possibility that shear stress in the bolts may have contributed to the initial failure. The tangential thrust at the seam in question is approximately 125 kips per ft of pipe, or 15.6 kips per bolt. It is possible that the initial torque to which the bolts were subjected during construction may have created enough

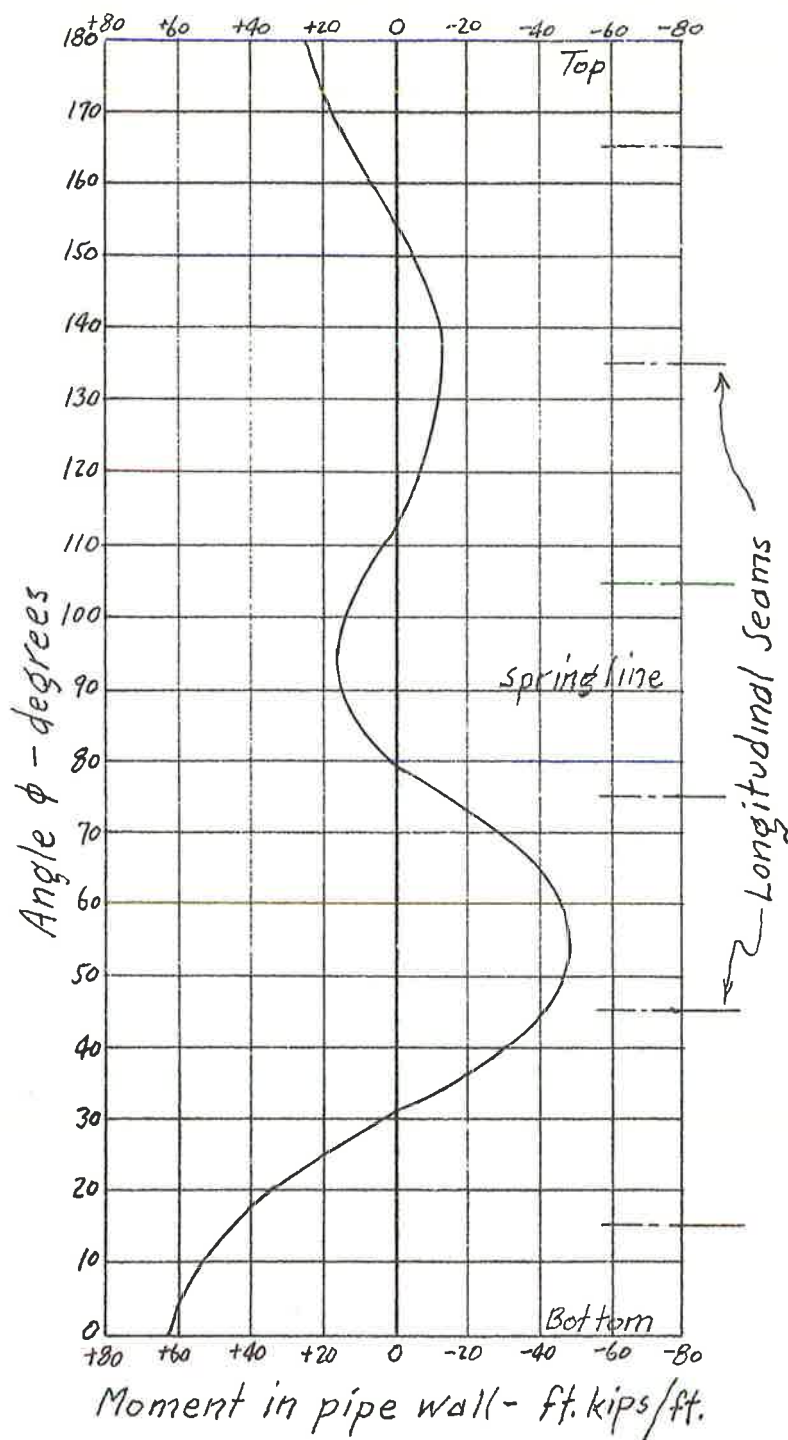


Figure 2. Moment diagram—Wolf Creek culvert.

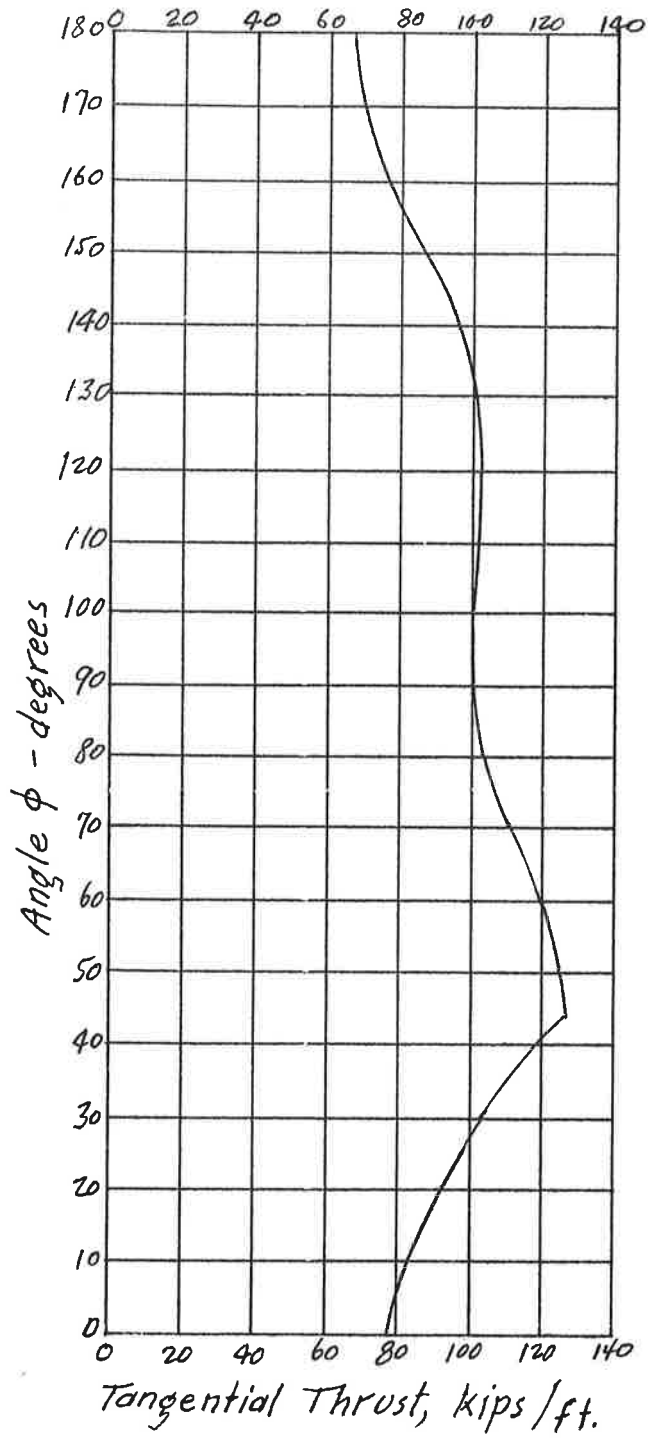


Figure 3. Thrust diagram—Wolf Creek culvert.

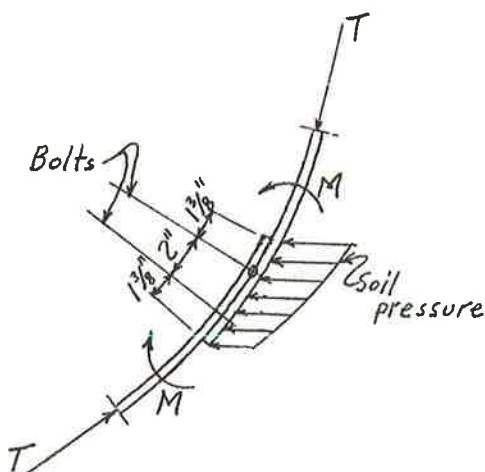


Figure 4. Forces on bolted seam—Wolf Creek culvert.

frictional resistance between the plates to carry this amount of thrust without stressing the bolts in shear. However, as bending moment developed, this frictional resistance would be relieved by the prying action of the moment. The magnitude of this relief is not readily determinable. If this transfer of shearing resistance from friction between the plates to shear stress in the bolts occurred, the composite effect of shear and tension in the bolts would have entered into the bolt failure phenomenon. Some recent research at the University of Illinois (1) has contributed materially to understanding of the failure of high strength bolts under composite stress conditions.

An important result of this analysis is the demonstration that the maximum stressed longitudinal seams are located in the quadrants below the springline (Figs. 2, 3). This is the region of extensive bolt failures as shown in Figure 2 of the Macadam paper.

In the above analysis it was assumed that the load on the pipe was equal simply to the dead weight of the prism of soil directly above the structure. This is tantamount to assuming that the product of the projection ratio and the settlement ratio was equal to zero (5, Chap. 24). The projection ratio is defined as the ratio of the distance which the top of the pipe projects above the natural ground surface to the horizontal dimension of the pipe. It apparently was about 0.9 in this case, as indicated in Figure 11 of the Kraft and Eagle paper.

The settlement ratio is a function of the relative settlement between the top of the pipe and the critical plane of the soil fill at the sides of the pipe, i. e., the horizontal plane which was originally level with the top of the pipe when construction of the fill began $H = 0$. If the critical plane settles more than the top of the pipe, shearing forces develop along the vertical sides of the central prism of soil, which are directed downward. The settlement ratio in this circumstance is positive and the load on the pipe is greater than the weight of the central prism.

If the critical plane settles less than the top of the pipe, the settlement ratio is negative, the shearing forces are directed upward, and the load on the pipe is less than the weight of the prism. A neutral or transition case occurs if the top of the pipe and the critical plane settle the same amount. The settlement ratio equals zero, there are no shearing forces, and the load on the pipe is simply equal to the weight of the central prism of overburden soil.

Some field measurements of settlement ratios of highway culverts were made by the writer prior to World War II (4). Most of the culverts studied were of the rigid type, but measurements were made on 4 corrugated steel culverts. Of these, the measured values of settlement ratio of 3 of the pipes were practically zero. However, on the fourth pipe the measured value was +0.8. This is a very small sample and general conclusions relative to settlement ratios which develop in connection with flexible culverts are not justified. However, they do indicate that under some circumstances, the settlement ratio for a flexible pipe installation can be a relatively high positive value, and the load on the pipe can be substantially greater than the weight of the overlying prism of soil.

If this situation prevailed in the Wolf Creek installation, the conditions of load and supporting strength analyzed above for an 83-ft hill could have developed at fill heights considerably below 83 ft. To illustrate, assume settlement ratios of +0.22 and +0.78 (products $r_{sd}P = +0.2$ and +0.7). The estimated load of 201 kips per lineal ft might have

occurred at fill heights of 63 ft and 54 ft, respectively, under these assumed circumstances.

Kraft and Eagle state that an imperfect ditch environment was created during reconstruction of the culvert by placing a layer of baled straw to a depth of 5 to 7.5 ft above the pipe and a 6-ft layer of loose soil above the straw. Also, $\frac{7}{8}$ -in. ASTM A 325 bolts were used to replace the $\frac{3}{4}$ -in. A 354 bolts. The writer estimates that the imperfect ditch construction will reduce the vertical load on the pipe from 201 klf to about 139 klf, or 31 percent. The strength of the bolts in tension will be increased from 50.1 to 55.5 kips, or nearly 11 percent. It is probable that the reconstructed culvert will safely carry the load of the embankment, but the factor of safety of the bolted seams will be substantially less than 3.5, the factor which was intended for the original design.

Conclusions

The writer's analysis of the structural failure of the Wolf Creek culvert leads to the following conclusions:

1. Failures of the longitudinal bolted seams were caused by a combination of using a design load which was too low, plus inadequate strength and spacing of bolts to carry the composite direct tension and possible shear stress to which they were subjected.

2. The designers state that the unit weight of the overlying embankment material "... could be expected to be 130 lb per cu ft or more." However, they used a unit weight of 105 pcf in calculating the design load, whereas the Marston Theory of loads on underground conduits indicates that the actual unit weight should be used.

3. The culvert was designed by the Ring Compression Theory. A major weakness of this theory is its failure to recognize the existence of bending moment in the pipe wall. Actually, there is substantial bending moment around the periphery of a flexible pipe culvert. The function of a longitudinal seam in the wall is to transmit both the bending moment and the tangential thrust from one ring plate to the next.

4. Calculations of direct tensile stress in the seam bolts due to prying action accompanying transmittal of bending indicate that the bolts were stressed beyond their ultimate strength. Failure of the bolts was inevitable.

5. Both bending moment and tangential thrust are greater in the lower quadrants. This accounts for the fact that practically all bolt failures occurred in the lower quadrants.

6. The designers used a settlement ratio equal to zero in calculating the design load. There is a possibility that the settlement ratio may have been greater than zero, which would have increased the load beyond the dead weight of the prism of embankment material directly over the culvert.

7. Lack of knowledge concerning the actual settlement ratio plus uncertainty relative to the action of a 2-row bolted seam in transmitting bending moment, lead the writer to believe it is probable that failure of the bolted seams began at a height of fill substantially less than the final height of 83 ft. The height at which failure began may have been as low as 50 or 60 ft.

8. The utilization of the imperfect ditch method during reconstruction of the embankment over the repaired culvert, plus the use of $\frac{7}{8}$ -in. A 325 bolts apparently will produce a safe structure. However, the factor of safety will probably be substantially less than the factor of 3.5 aimed at in the original design.

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JOHN N. MACADAM, Closure—The author appreciates Rumpf's concern for possible interpretations of a paper which might prejudice engineers against the use of A 490 bolts for structural connections. The paper was concerned only with "A 490-type" bolts used in structural plate culvert pipe, which is a limited and perhaps unique application. It should be strongly emphasized that neither the geometric configuration nor the field application of these bolts corresponds to traditional high strength bolting uses, i. e., building and bridge construction. The conclusions reached in the paper must not be interpreted as dissatisfaction with the performance of high strength bolts used for conventional structural joints, either friction or bearing-types. The good performance of such joints is a matter of common knowledge gained over a period of years. The work of the Research Council on Riveted and Bolted Structural Joints has greatly contributed to the favorable usage of high strength bolts.

It is encouraging to know that the Research Council has recently initiated investigations on problems of stress corrosion of high strength bolts and also on galvanized bolts. Through the auspices of organizations such as the Research Council, a better understanding of structural bolt behavior in corrosive applications can be attained.

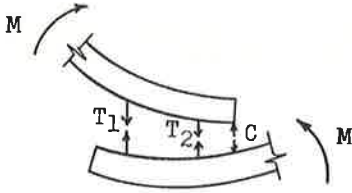
Spangler's remarks are directed jointly to the author's paper and the companion paper by Kraft and Eagle. Spangler's study of the field failure is a worthy contribution to the papers. Although a hydrogen-stress cracking hypothesis is probably uncommon to those concerned with highway culvert performance, this type of phenomenon in high strength steel has been known for some time and is well documented (1, 2, 3, 4). It is known that a level of sustained tensile stress and a hardness comparable to the failed bolts at Wolf Creek, together with a source of nascent hydrogen, can cause failure.

Spangler has concluded that the A 490 bolts in the longitudinal seams failed due to over-stressing in a combination of direct tension from transferal of bending moment in the pipe wall and shear from thrust transferal. In the chronological development of the author's investigation, static strength tests were made initially in an effort to explain the field failure by a mechanical weakness of the bolts or an overstressed state. It was only after the static load experiments failed to resolve the failures that environmental effects were investigated.

As pointed out in the paper, only A 490 bolts failed. For all practical purposes the failures extended up to locations in the pipe where A 325 type bolts were used. If overloading had been the cause of failure, it would seem likely that some A 325 bolts adjacent to the A 490 bolts would have also failed, and in a ductile manner. Bolt fracture surfaces generally resembled the brittle fracture surfaces from the laboratory hydrogen-stress cracking experiments more than typical combined shear-tension failures.

Using all of Spangler's assumptions for the conditions at Wolf Creek, together with his well-known method of elastic analysis, resulted in a moment of 480 in.-kip per ft for the critical longitudinal seam. However, the maximum moment capacity of the $\frac{3}{8}$ -in. structural plate by tests is only 143 in.-kip per ft, as given in Table 3 of the paper. Spangler's computed value of 480 in.-kip per ft is therefore a moment which cannot be developed by the pipe wall. Hence, his deduction that 60 kips per bolt would be developed in direct tension does not follow.

A rational analysis of the bolted lap seam subjected to moment is shown in Figure 1. For a moment of 143 in.-kip per ft acting on the seam, bolt tensions of 9,200 lb/bolt and 3,400 lb/bolt are obtained for the outer and inner bolt rows, respectively. This analysis is based on a condition of no initial bolt tension from tightening. Actually, with



From ΣM equation and compatibility of deformations equation the bolt tensions T_1 and T_2 can be determined:

$$3.375 T_1 + 1.375 T_2 = M$$

$$T_1 = 3.109 T_2 - 0.0372 M$$

$$T_1 = 0.2577 M$$

$$T_2 = 0.09483 M$$

for $M = 143,000$ in.-lb per ft

$$T_1 = 36,800 \text{ lb for 4 bolts or } 9,200 \text{ lb/bolt}$$

$$T_2 = 13,600 \text{ lb for 4 bolts or } 3,400 \text{ lb/bolt}$$

Figure 1. Analysis of $\frac{3}{8}$ -in. structural plate lapped seam subjected to moment.

initial prestress in the joint, the bolt tensions are for all practical purposes unchanged until the tension from external loading exceeds the initial tension.

The combination of 9,200 lb maximum possible direct tension and Spangler's value of 15,600-lb shear can be compared to the results obtained by Chesson et al. (5). Figure 2 shows the combined load state for the structural plate bolts as related to an interaction curve from tests on $\frac{3}{4}$ -in. A 354 BD (A 490) bolts with threads in the shear plane. A factor of safety against failure of approximately 1.7 exists.

It is recognized that the ring compression method predicts that installed culvert pipe performance can be based on pipe wall thrusts. Some bending moment does exist in the pipe wall, but a history of properly backfilled installations has shown that the ring compression method is a successful design approach. Spangler's method of analysis for computing bending moments apparently can sometimes result in values which cannot be resisted by the pipe wall. In the case of the Wolf Creek structure, such computed moments lead to a false hypothesis of the bolt failures.

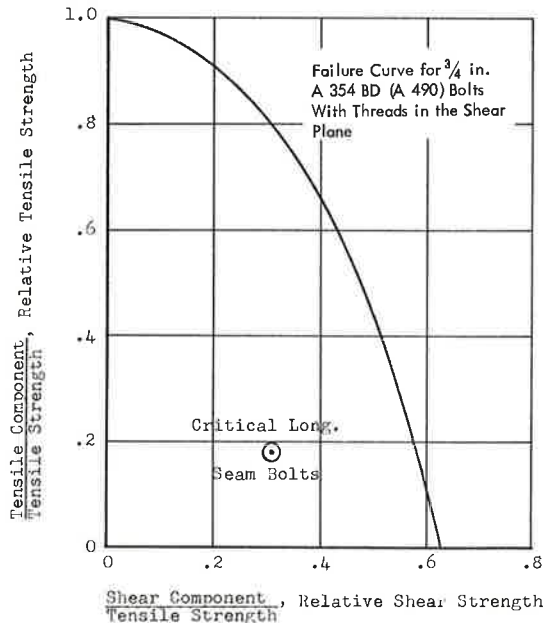


Figure 2. Comparison of critical longitudinal seam bolts and interaction failure curve from Illionis tests (Fig. 4 of Ref. 5).

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HAROLD L. EAGLE, Closure—Professor Spangler implies that from his experience the earth load on a buried pipe in a rock fill can be expected to be as large as the weight of the entire soil mass directly over the pipe. The design of the original pipe did recommend the use of an imperfect trench fill. In reviews of the design, this recommendation was eliminated as being unnecessary. The rebuilt pipe has been constructed with the imperfect trench method using a 30-in. layer of hay a short distance above the pipe and the same width as the pipe. This fill has been completed for over 3 months. Instrumentation of the installation by Montana State University is functioning quite well. This instrumentation indicates roughly 60 percent of the weight of the fill directly over the pipe is presently being supported by the pipe. We expect there will be some increases in this percentage in the years ahead. We admit construction without the imperfect trench method would be expected to produce greater loads on the pipe. However, as presently constructed, it is our opinion the load on the pipe will remain substantially less than the entire weight of the fill over the pipe. This was the original basis for design.

Professor Spangler in his discussion considers a vertical deflection of the pipe of 20 in. when failure occurred. Evaluation of the pipe dimensions after failure for the entire length of the pipe indicates the vertical deflection at the time of failure was about one foot, and that the increase from initial shape in horizontal diameter was about 0.6 ft. We made moment calculations for the indicated distorted shape assuming an entirely "elastic" condition. This analysis did indicate a moment of about 27 kip ft per ft near the lower quarter points and higher moments at the bottom. However, evaluation of the yield moment of the pipe section for a "plastic" condition using a steel stress of 35,000 to 40,000 psi reveals that the pipe shell cannot resist a moment of over 11 to 13 kip ft per ft. Actual bending tests have shown the plastic hinge moment for the section to be 11 to 12.5 kip ft per ft.

Professor Spangler's consideration of a moment of 63 kip ft per ft is therefore not valid. The deflection assumed is about twice that which occurred at failure and plastic hinges in the pipe shell limited the maximum moment to about 11 or 12 kip ft per ft.

We did make a theoretical analysis of bolt stresses including direct tensile stresses, torsional stresses, shear stresses, and bolt tensile stresses due to bending in the joint. When these stresses are combined and maximum tensile and shear stresses computed, values are obtained approaching yield point stresses for the high strength bolt material. We do not feel these theoretical analyses tell the whole story. We have used hot driven rivets successfully for years, yet every good rivet reaches its yield stress in cooling. The rivets still carry large added shear and tensile loads. We have concluded that actual joint tests are essential to evaluate joint adequacy.

Mr. Macadam reports on extensive physical testing of the bolted joints in his paper. Figures 8, 9, 10, 11 and 12 in that report show photographs of shear and bending tests

which were constructed with varying amounts of bolt torque. We believe Macadam's statement is well founded by the laboratory tests of joint strength: "No evidence of bolt or joint weakness was observed in any of the static strength tests made in the laboratory in search of a plausible explanation of field failure."

Conclusion

I do agree with Prof. Spangler that the structural plate pipe may have been subjected to greater than design loads. If so, based on data received from instrumentation of the rebuilt pipe, any overload which occurred was probably due to the elimination of the imperfect fill method recommended by the original design. I strongly disagree with Spangler's conclusion that longitudinal seam strength was too low. The physical testing of the joint has demonstrated that normally the pipe shell would yield prior to a joint failure. I feel that the hydrogen-stress cracking explanation set forth in Macadam's paper does appear to be a major factor to consider in evaluating the failure of the Wolf Creek Pipe.