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Welded Aluminum Highway Structures

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This paper discusses the applications of welded aluminum in highway structures from the standpoints of alloy recommendations and mechanical properties of the material, structural members, design features and specifications, welding processes, welder qualification, and precautions and methods in field welding.

•THE YEARS 1958, 1959, 1960, and 1961 are milestones in aluminum's long history. Old consumption records fell by the wayside and new total consumption marks were established in each of those years.

The building products industry was particularly outstanding. The official 1960 shipments of the aluminum industry to that field were 565,000 tons. Shipments to users in the building products industry were about 25 percent of total shipments, larger than any other industry. Official figures for 1961 are optimistically awaited. This growing industry fully expects to reach new heights in the year just beginning, 1962.

The use of aluminum in the highway field also made great strides from 1958 to 1961. In most of those years, the consumed tonnage doubled the figures for the previous year. The industry readily admits that the acceleration of the nations's highway programs on Federal, State, county, and city levels was a terrific force behind this upsurge. However, expanding recognition of aluminum's many natural advantages was undoubtedly influential in establishing the new records.

ADVANTAGES

Aluminum's advantages are threefold: (a) freedom from maintenance, (b) lightweight, and (c) versatility of form. These features will be fully explored to present a complete understanding of each.

Freedom from Maintenance

The natural formation of a tough, colorless oxide on the surface of the aluminum when exposed to air is the prime factor in aluminum's maintenance-free characteristics. This complex oxide layer protects the bulk of the metal from structural deterioration and rapidly seals itself if scratched. Of course, the degree of protection varies, depending on the alloy composition. It is important that specifications for structural aluminum alloys require the use of the more corrosion-resistant alloys or suitable protection for the less resistant alloys.

All the commercially weldable alloys and their recommended filler wires have a high degree of corrosion resistance and never need to be painted in highway applications. Generally speaking, these alloys are of the 3000, 4000, 5000, and 6000 series. Because they are not readily weldable, 2000 series alloys are frequently used in mechanically joined assemblies. Anodizing, cladding, and painting maintain the structural integrities of the 2000 series of alloys in corrosive environments.

Aluminum alloys that are recommended for welding never need painting. It is true that the structures will not maintain their "ribbon-cutting day" gleam indefinitely. The surfaces may become dirty in all but the purest environments, and self-limiting surface corrosion may occur in severe industrial and coastal atmospheres. Inspectors may assume, however, that these surfaces and those unseen to the eyes are not suffering any significant loss in their structural integrity.

Aluminum's corrosion resistance eliminates the maintenance crews' paint brushes and the related costs of the painting operation. The financial burden of completing the road-building job demands that limited funds must not be misdirected toward needless maintenance operations.

The safety of maintenance personnel and the motorists must be guaranteed. It is obvious that the removal of workmen and their necessary trucks, scaffolds, and ladders are instrumental in promoting this requirement. The public relations advantage can be readily appreciated, not to mention the hundreds of thousands of dollars saved in potential law suits.

No discussion would be comple without consideration of the compatibility of aluminum with other construction materials. Although aluminum alloy fasteners are recommended, stainless steel and zinc- or cadmium-plated steel fasteners are used with aluminum components without fear of galvanic corrosion in all but the most severe environments. Large faying surfaces between aluminum and any ferrous material should be protected. In these cases, the steel should be coated with red lead paint or iron oxide primer followed by an application of aluminum paint. The aluminum should be protected with a coating of zinc chromate primer. Aluminum in contact with wood or earth should receive a heavy coat of a chemical-resistant bituminous paint. More detailed information on these suggested paint systems is found elsewhere (1, Sec. I-6).

The Alcoa Research Laboratories (2) state that aluminum parts imbedded in structural concrete need not be painted to maintain their structural integrity.

Lightweight

The density of aluminum is one-third that of steel. However, the lower allowable stresses for aluminum dictate that the aluminum components shall have larger cross-sections than their steel counterparts. This means that the dead weight ratio of the two materials in the fabricated structure is about 2:1. This 50 percent weight reduction is significant for a number of reasons.

The most important benefit is the possibility of designing the structure for lighter dead loads. In extemely large bridges, where the dead weight is a large portion of the critical design load, this advantage can drastically affect the economics of the structure. Movable structures, such as bascule or lift spans, have been designed with lighter counter-balances because aluminum was used. Notable examples are the Hendon Dock Junction, Sunderland, England, bridge and the Victoria Dock, Aberdeen, Scotland, bridge. Both of these double-leaf trunnion bascule bridges used standard aluminum structural shapes and plate.

Larger shop-fabricated aluminum components are other benefits of the lightweight feature. Aluminum's lightweight encourages the use of large shop assemblies which are particularly well-adapted to joining by welding.

Expensive, and sometimes erratic, field fabrication is held to a minimum. A prime example of this feature is the 220-ft welded plate girder span erected in 1958 near Des Moines, Iowa (Fig. 1). Assemblies consisting of two girders and diaphragms were completely shop fabricated by Pullman Standard Company, Chicago, Ill., and shipped on flatcars and trailers to the erection site. Field erection was held to a few shear connections between diaphragms and girders and several girder splices.

Shipping and erection costs are held to a minimum with aluminum. Shipping expenses are drastically sliced because of the 50 percent weight saving. Many aluminum highway items, such as signs and traffic signal posts, are erected without need for mechanical lifting equipment (Figs. 2 and 3). Obviously, bridge components require mechanized equipment, but its capacity, and therefore its cost, is lowered (Fig. 4).

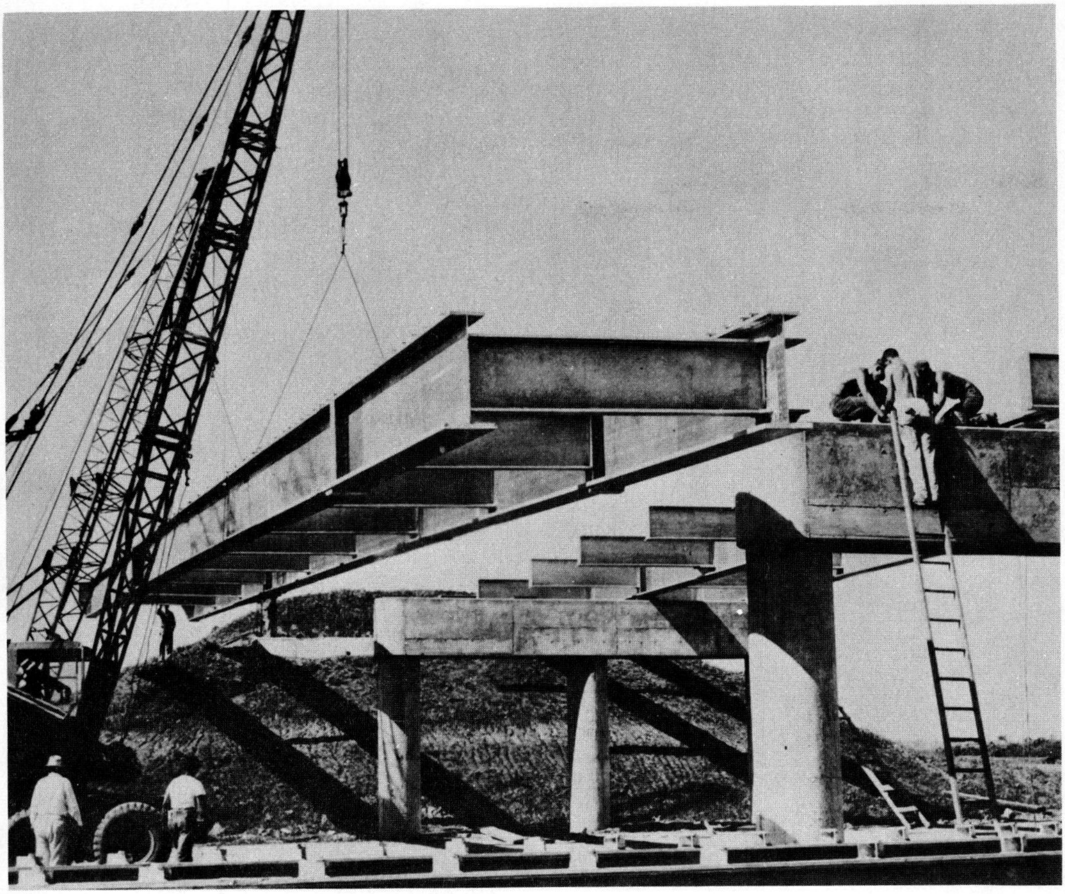


Figure 1. Giant subassembly of aluminum structural members elevated into position during construction of first aluminum girder-type highway bridge in the world, located near Des Moines, Iowa. Quick and easy field erection made possible by prefabrication of relatively lightweight members. Pioneering project jointly sponsored by Iowa State Highway Commission and Aluminum Company of America, Kaiser Aluminum and Chemical Corporation, and Reynolds Metals Company.

Versatility of Form

A vast array of aluminum products is available to the designer. Sheet and plate are available in thicknesses up to 3 in. and in widths up to 136 in. Lengths up to 45 ft are available in the common thicknesses of plate. Standard structural shapes are produced by rolling or extruding. Sizes up to 10- by 10- by $\frac{1}{4}$-in. angles, 12-in. I-beams, and 15-in. channels are available and the more common sizes are produced in lengths up to 85 ft.

Deeper composite plate girders, employing moderate-strength alloy webs and high-strength alloy flanges, may be designed and fabricated. As an example, a welded plate girder may have a 6061-T6 web and 5456-H321 flanges. This configuration makes the most of the mechanical properties of aluminum alloys.

The versatile extrusion process allows the designer additional freedoms in developing an efficient aluminum structure. Intricate single shapes may be substituted for built-up assemblies and special projections or lugs may be integrated to facilitate special joining techniques. As an example in the welding field, integral back-up strips and bevels eliminate some joint preparation operations.

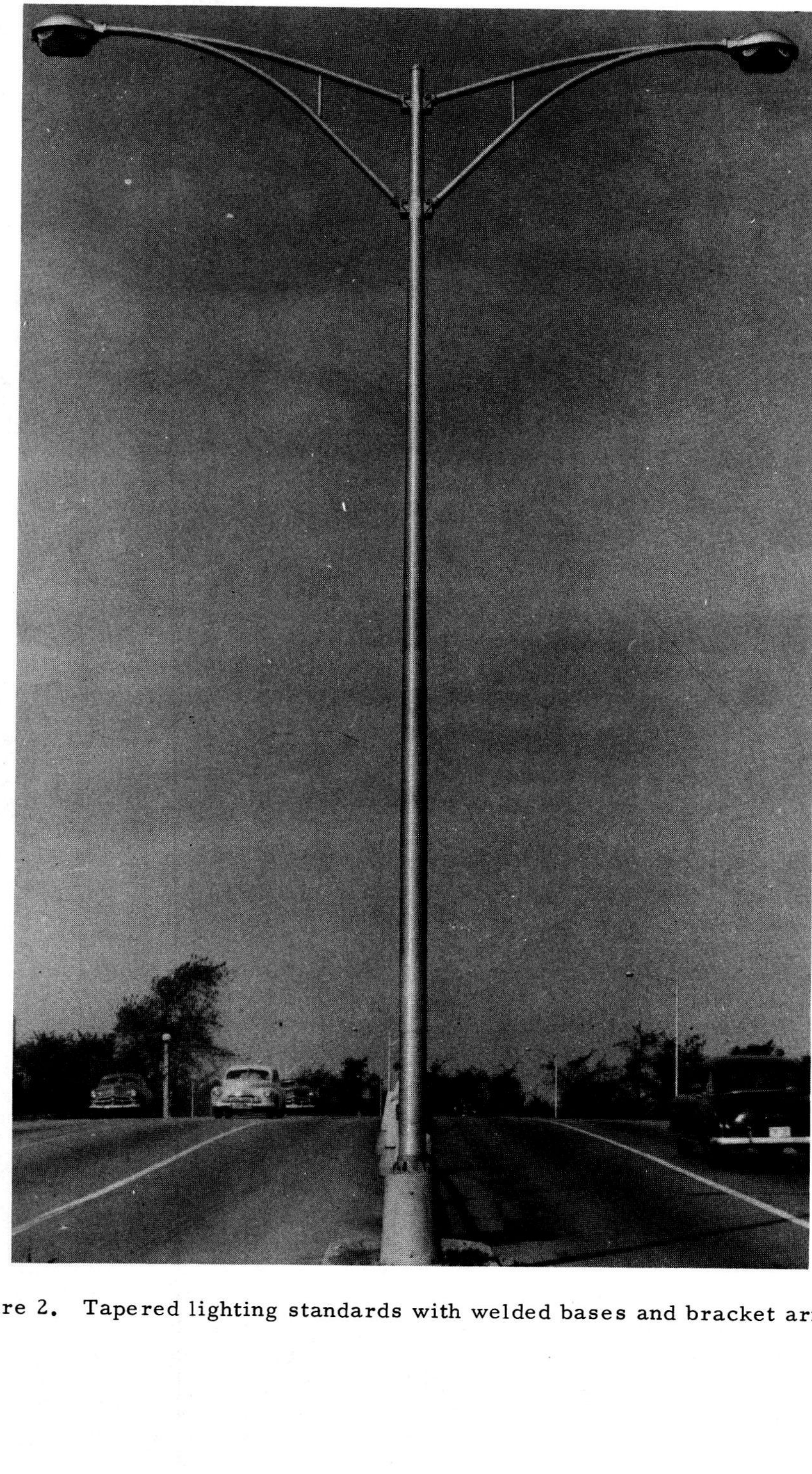


Figure 2. Tapered lighting standards with welded bases and bracket arms.

An extrusion press with the largest capacity in the United States, 14,000 tons, is owned and operated by Alcoa. This press is identical to one owned by the U. S. Government and operated by Alcoa which has produced huge extrusions for the government aircraft and missile programs for many years. This press is capable of producing extruded shapes up to 39 in. in width and up to 110 ft in length.

ALLOY RECOMMENDATIONS

The most common structural aluminum alloys are 6061-T6 and 6062-T6. These alloys are particularly well suited to mechanically joined structures, but welded fabrication is frequently employed. A minimum tensile strength of 24,000 psi across a butt joint is delivered by these alloys. Recognized design stresses for 6061-T6 and 6062-T6 are presented in ASCE Paper 970. When these alloys are used in sign structures, the working stresses shown in the AASHO Specifications for the Design and Construction of Structural Supports for Highway Signs (3) should be followed.

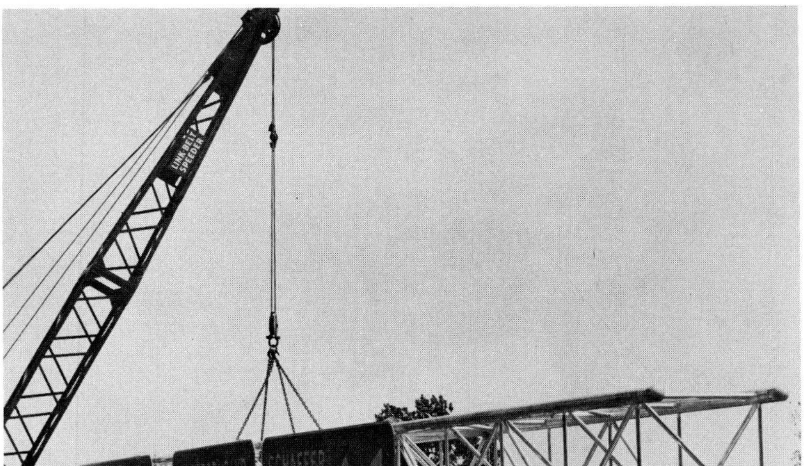


Figure 3. Welded tubular truss supporting overhead highway signs.

Alloy 5456 holds great promise for welded structural applications. Its use in highway structures has been limited to date, but its minimum tensile strength across a butt joint of 42,000 psi demands increased consideration. Working stresses are contained in ASCE Paper 2528 (4).

Alloy 6063-T6 is frequently used in low stress applications. Its 17,000-psi minimum tensile stress across a butt welded joint is often satisfactory in structures that are controlled by rigidity limitations. Design stresses for structural applications are anticipated from ASCE. The AASHO sign structure specifications (3) contain working stresses for alloy 6063-T6.

APPLICATIONS

Bridges

The history of aluminum bridges dates back to 1933 when the Smithfield Street Bridge in Pittsburgh, Pa., was refurbished with an aluminum deck. Riveting, the most popular joining method in that day, was used to assemble that structure.

Welding now has become established as a recognized joining technique. Bridge designers are urged to consider this technique for their modern structures.

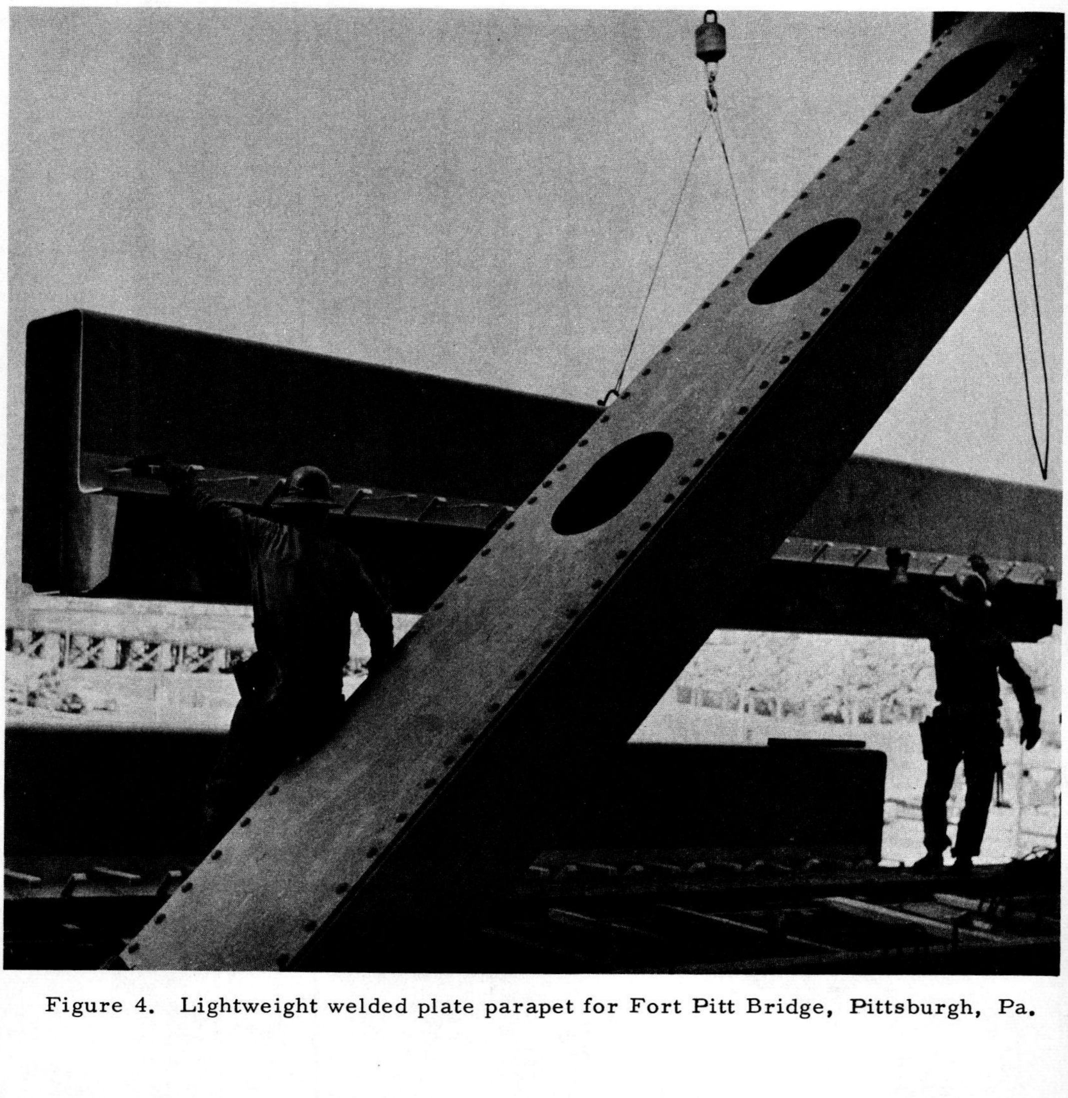


Figure 4. Lightweight welded plate parapet for Fort Pitt Bridge, Pittsburgh, Pa.

A newsworthy application in recent years is the previously mentioned 220-ft welded plate girder span erected near Des Moines, Iowa, in 1958. This four-span continuous structure uses alloy 5083-H113 plate and alloy 5183 filler wire. These alloys are members of the aluminum-magnesium alloys family, which retain a great degree of their original strength after welding. The girders at the critical sections are composed of 36- by ½-in. web plates, 12- by ¾-in. top flanges, and 12- by 1-in. bottom flanges. Eight-foot widths of the 8-in. reinforced concrete deck at each girder are depended on for composite action.

Ned L. Ashton of Iowa City, Iowa, was the consultant for the Iowa State Highway Commission and the aluminum components were fabricated by the Pullman Standard Company of Chicago, Ill. The bridge was designed for H20-S16 loading and carries a two-lane highway over a four-lane divided section of an Interstate highway.

The bridge used only 75,000 lb of aluminum alloys, and the aluminum items were bid at $1.00 per lb fabricated and erected.

Overhead Sign Structures

Proper signing increases the efficiency of modern highways. The most effective location to mount these vital signs is above the highway on overhead sign structures. Aluminum is enjoying great usage in this application.

The Michigan State Highway Department was the leader in this aluminum development. Many other State highway departments and city and county agencies now recognize aluminum's advantages and accept it as a standard construction material. Hundreds of tons of aluminum are used annually in this application.

Most of these structures are fabricated from alloy 6062-T6 extruded tubes which are welded into three- or four-chord trusses with alloy 4043, 5356, or 5556 welding filler wire. The widths of these trusses range from 2 to 7 ft and the depths are in the 3- to 12-ft range. Lengths of 100 ft are common and many of the newer spans cover 160 ft. The trusses are fabricated in 25- to 50-ft long modules which are bolted together at the erection site. Two chord posts are fabricated in single units about 25 ft long. Typical chords are 4- to 6-in. OD-extruded 6062-T6 tubes, and the web struts are normally about 2½-in. OD-extruded tubes.

Recent AASHO Specifications for the Design and Construction of Structural Supports for Highway Signs (3) cover the design of aluminum overhead sign structures. The aluminum industry welcomes this standardization and anticipates that it will begin a new era of even greater acceptance of this material.

Roadside Sign Structures

Roadside signing supplements the larger overhead mounted signs. Aluminum also is recognized in this application.

Straight and tapered round tubes are favorite types of poles because of their appearance and torsional strength advantages over structural shapes. The bases of these alloy 6062-T6 or 6063-T6 tubes, which are normally about 8 in. OD, are welded to alloy 356 castings or 6061-T6 plates. These bases are bolted to aluminum or galvanized steel anchor bolts which are imbedded in concrete pedestals.

Signs

Aluminum's corrosion resistance makes it a natural for directional and informational signs. Most of these signs are bolted assemblies of special extruded channel-shaped sections or flat 0.125-in. sheet. An exception is a recently developed panel which is a resistance-welded assembly of extrusions and sheet.

These new panels are available in 12- to 36-in. widths which help to reduce the number of joints between panels. The facing sheet is alloy 3003-H18 in an 0.080-in. thickness. Alloy 6063-T6 extrusions with 2-in. depths and integrally extruded bolt head slots are welded to the facing sheet for additional strength and rigidity. The intermittent seam welding process is used for making these joints. The dimples on the facing sheet are slight and do not adversely affect the performance of any reflective surface or paint coating. Special hardware makes attachment to post flanges a simple job.

Lighting Standards

All lighting standards of aluminum depend on welding for their strength and rigidity.

Specially curved bracket arms of alloy 6063-T6 extruded tubes are normally shop welded to brackets that are field bolted to the poles. These poles are either spun-tapered alloy 6063-T6 round tubes, alloy 5052 flat sheet formed into round shafts and then longitudinally seam welded, or brake-formed alloy 6061-T6 hexagonal shafts. The shafts are slipped inside cast bases and alloy 4043 or 5556 continuous fillet welds are placed between the two.

There is no need for engineers to be concerned about originating special lighting standard designs for each project. A number of well-qualified fabricators have a myriad of pre-engineered designs. Their engineers are willing to work with highway engineers to develop special designs if needed.

Traffic Signal Pedestals

The construction of these items is similar to that used for lighting standards. Alloy 6063-T6 spun-tapered tubes are fillet welded to alloy 356 bases.

These poles have been used by New York City and other municipalities for a number of years. The principal advantage of aluminum is its lightweight. Initial installation and replacement costs are drastically reduced because of this feature.

Bridge Railing

Many wrought bridge railings are assembled by shop welding. Common-welded joints in these assemblies are between the balusters and rails and between the posts and base plates.

The balusters are solid bars of alloy 6061-T6 or extruded tubes of 6061-T6 or 6063-T6. Rails and posts are normally 6061-T6 or 6063-T6 extruded shapes or tubes. Filler wires are alloy 4043 of 5556.

SUMMARY

Welded aluminum highway structures have good service records and their popularity is increasing. Engineers are acknowledging aluminum's advantages: (a) freedom from maintenance, (b) lightweight, and (c) versatility of form. Bridges, overhead sign structures, roadside sign structures, signs, lighting standards, traffic signal pedestals, and bridge railings make frequent use of welding.

DESIGN

Effect of Welding on Mechanical Properties

Most of the structural aluminum alloys attain their strength by heat treatment or strain hardening. The heat of welding removes part of the effect of the prior heat treatment or strain hardening in the vicinity of the weld, causing this heat-affected material to be weaker than the unaffected metal. The resulting effect on mechanical properties in the vicinity of a weld is illustrated by the typical distribution of yield strength shown in Figure 5. The distance b_h in the figure designates the extent of the "heat-affected zone," measured from the center of the weld.

In evaluating the effect of a weld on the strength of a member, it is convenient to consider that the material in the vicinity of a weld has only two strength levels, as indicated by the dotted line. The material in the "reduced strength zone," whose extent is indicated by b_r, is considered to have minimum properties, and the material outside this zone is considered to have the properties of unaffected parent metal. The width of the zone is adjusted so that conservative values of the strength of members with longitudinal welds can be calculated as the sum of the strength of the material within the reduced strength zone and the strength of the unaffected metal outside this region (4).

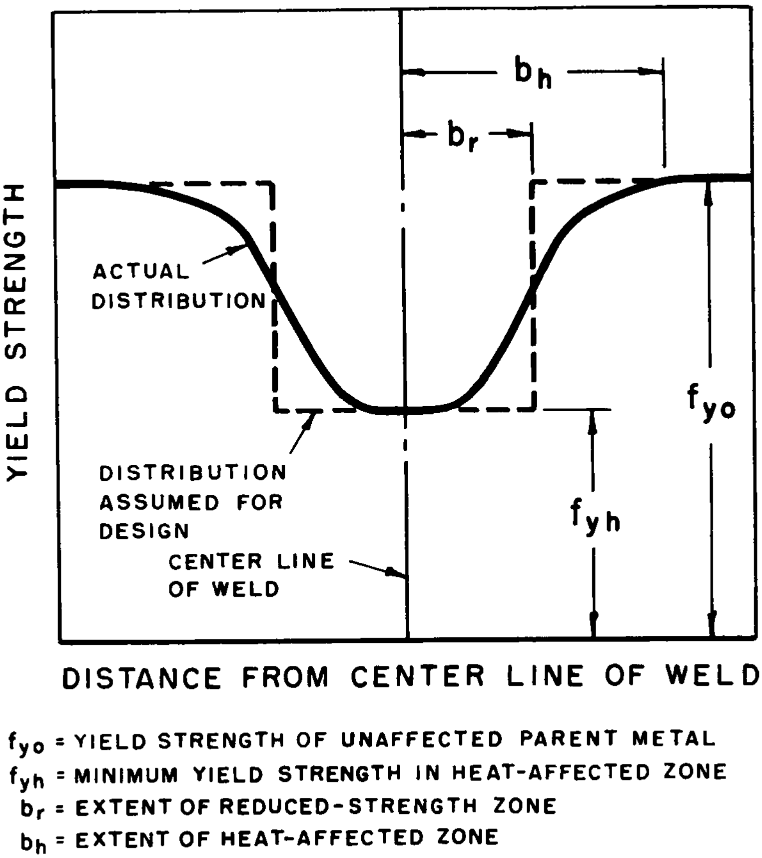


Figure 5. Typical distribution of yield strength values in vicinity of weld.

The reduced strength zone is considered to extend an equal distance in all directions from the center of a butt weld or the heel or a fillet weld. When the welding procedure is carefully controlled to minimize the heating of the material in the vicinity of the weld, the extent of the reduced-strength zone may be very small, approaching zero in some cases. These small values can be used for design in cases where they have been well established by hardness surveys or tension tests. However, for general design purposes in cases where it is not practical to determine the actual extent of reduced-strength zone, a value of 1 in. can be used for b_r.

Strength of Butt Welds

In the strain-hardened alloys, tensile strengths across butt welds equivalent to the strength of annealed material are developed (7, 8, 9). This strength is the basis for the weld qualification test requirements in the ASME Boiler and Pressure Vessel Code (6). In the case of heat-treatable alloys not heat treated after welding, weld strengths are intermediate between the strength of annealed material and the strength of the unaffected parent metal. The ASME welding qualification test requirements are 24,000 psi for 6061-T6 and 17,000 psi for 6063-T5 and -T6.

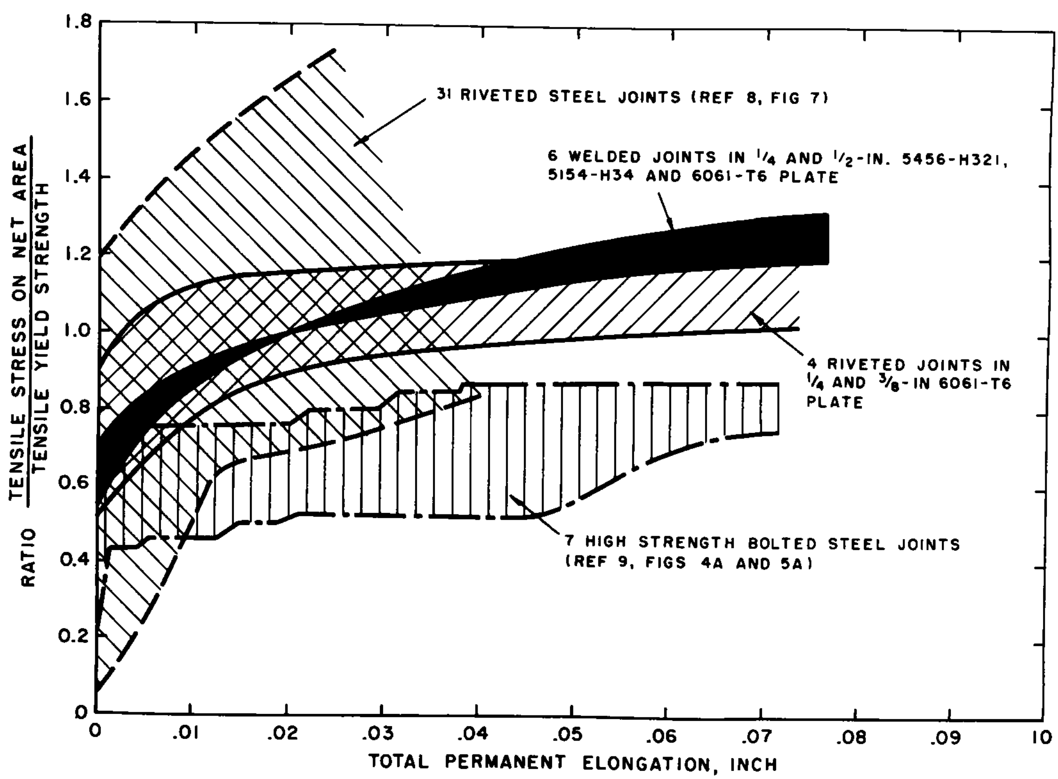
For welds that receive only visual inspection, it is recommended that a weld strength equal to 90 percent of the weld qualification test requirement value be used as the minimum expected weld strength for the purpose of selecting allowable stresses. In determining allowable stresses, this minimum expected weld strength should be divided by the same factor of safety that is applied to the ultimate tensile strength of the material in structures without welds. For welds stressed longitudinally, the minimum expected strength of the material in the reduced strength zone should be considered to be the same as the minimum expected transverse weld strength.

Because of heat-of-welding effects, the yield strength across butt welds made in strain-hardened or heat-treated aluminum alloys depends on the gage length used. The yield strength value corresponding to 0.2 percent offset on a 10-in. gage length is considered to be applicable to the design of welded structures (4). This offset represents a permanent deformation across a joint of 0.02 in., which, as shown in Figure 6, is typical of the deformation in riveted or bolted joints at loads that cause an average net section stress equal to the yield strength (12, 13, 14).

Values of minimum tensile strength and yield strength across butt welds for a number of weldable alloys are given in Table 1. Also, the table gives values of minimum tensile strength and yield strength for the weakest material in the reduced strength zone. Yield-strength values in this case were determined from specimens stressed parallel to the weld, so that gage length does not influence the yield strength.

Strength of Fillet Welds

It is recommended that the minimum shear strength of fillet welds for design purposes be taken as 75 percent of the average value determined from tests (7, 8). Values of minimum shear strength of fillet welds determined in this way are given in Table 1. The minimum expected tensile strength of the material immediately adjacent to the



NOTE: TENSILE YIELD STRENGTH IS STRESS AT 0.2 PER CENT OFFSET ON 10-IN. GAGE LENGTH ACROSS WELDED BUTT JOINTS AND YIELD STRENGTH OF MAIN PLATE MATERIAL FOR RIVETED AND BOLTED DOUBLE STRAP BUTT JOINTS

Figure 6. Permanent elongation of welded, riveted, and bolted joints.

TABLE 1

MINIMUM[1] STRENGTH DATA FOR WELDED JOINTS (TIG or MIG welding with no post weld heat treatment)

Parent Material(2)	Filler Wire (3)	Across Butt Weld: Tensile Strength, psi	Across Butt Weld: Yield Strength,(4) psi	In Reduced Strength Zone: Tensile Strength, psi	In Reduced Strength Zone: Yield Strength, f_{yh}, psi	Shear Strength of Fillet Welds(5), psi: Longitudinal	Shear Strength of Fillet Welds(5), psi: Transverse(6)
3003	1100(7)	14 000	7 000	14 000	5 000	8 000	9 500
3004	4043(7)(8)	23 000	11 000	23 000	8 500	11 500	16 000
5052	5652(7)(8)	25 000	13 000	25 000	9 500	13 500	19 000
5154	5254(7)(8)	30 000	15 000	30 000	11 000	16 000	23 000
5454	5554(7)(8)	31 000	16 000	31 000	12 000	16 000	23 000
5086	5356(7)(8)	35 000	19 000	35 000	14 000	17 000	26 000
5083	5556	40 000	24 000	40 000	18 000	20 000	30 000
5456	5556	42 000	26 000	42 000	19 000	20 000	30 000
6061-T6, 6062-T6	5556	24 000	20 000	24 000	15 000	20 000	21 000(10)
6061-T6, 6062-T6	5356	24 000	20 000	24 000	15 000	17 000	21 000(10)
6061-T6, 6062-T6	4043(7)(8)	24 000	15 000	24 000	11 000	11 500	16 000
6063-T5, -T6, -T83, -T831, -T832	4043(7)(9)	17 000	11 000	17 000	8 000	11 500	15 000(10)

(1) These are minimum expected strength values to be used as basis for design. Typical or average strength values are appreciably higher.

(2) Strength values apply to all tempers of nonheat-treatable alloys except as noted under note (4).

(3) Filler wires listed are commonly used. They do not necessarily represent recommended filler wires for all applications.

(4) Yield strength across a butt weld corresponds to 0.2 per cent set on a 10-in. gage length. Values listed for nonheat-treatable alloys are for tempers quarter hard or harder. For annealed temper, this value will be the same as yield strength in the reduced strength zone.

(5) Applicable to throat area of fillet.

(6) For transverse shear in unsymmetrical joints, such as single lap joints, use values for "Longitudinal Shear "

(7) Greater strengths, particularly in fillet welds, can be obtained with higher strength filler alloys.

(8) 5556 filler metal recommended for greatest strength.

(9) 5356 or 5556 filler metal recommended for greatest strength.

(10) Strength controlled by shear strength of parent metal adjacent to weld

welds should be considered to be the same as the minimum expected transverse strength of butt welds.

Fatigue Strength of Welds

Fatigue strength data have been reported for a wide variety of aluminum alloy joints (10, 11). Figure 7 shows typical values of fatigue strength determined in tests of butt welds at zero stress ratio. The effect of variation in stress ratio (that is, the ratio of minimum to maximum stress in a loading cycle) on fatigue strength is shown in Figure 8. These results illustrate the increase in fatigue strengths attributable to higher stress ratios often encountered in cyclic loading of structures. The curves shown in both Figures 7 and 8 represent the average test results for ⅜-in. thick plate specimens which were obtained from butt-welded panels prepared by welding, back-chipping, and filling the back-chipped groove. The panels were tested with the weld

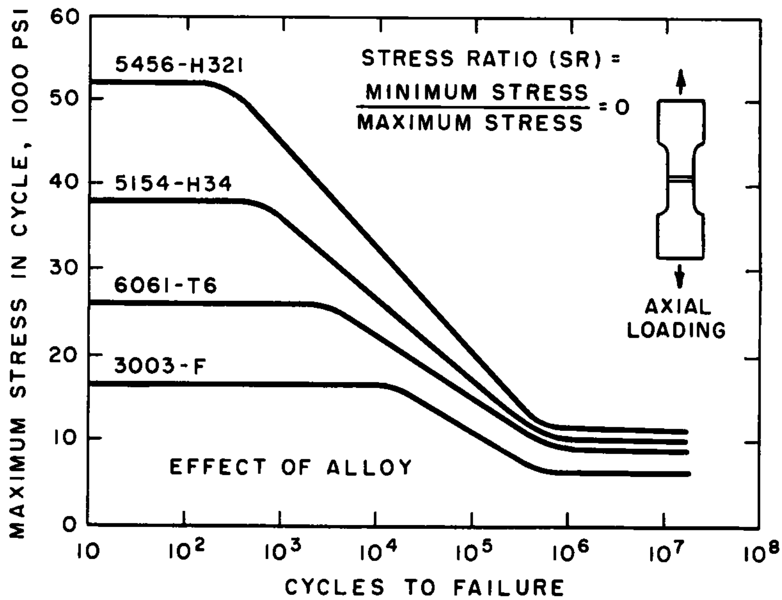


Figure 7. Direct stress fatigue test results for welded aluminum butt joints for inert-gas arc welds.

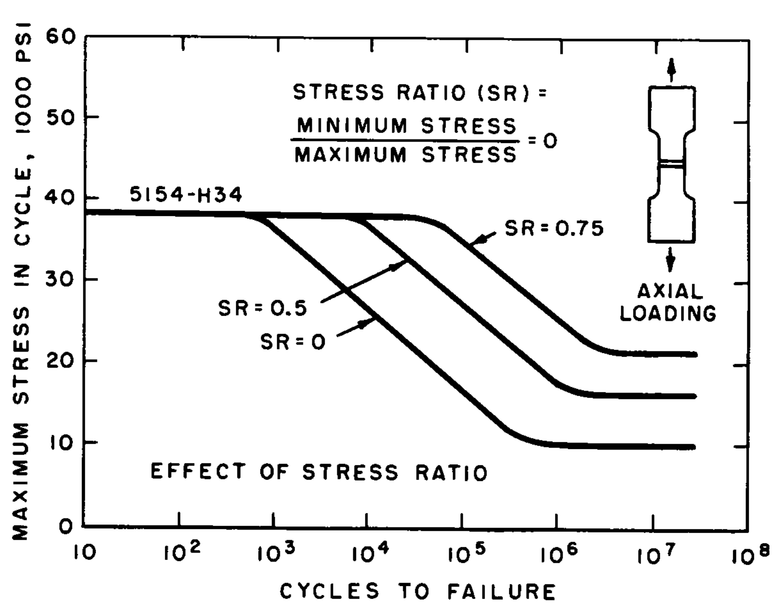


Figure 8. Direct stress fatigue test results for welded aluminum butt joints for inert-gas arc welds.

bead in the as-welded condition. The following additional observations are based on experience in fatigue-testing aluminum alloy joints:

1. The fatigue strengths of butt-welded joints generally equal or exceed those of a well-designed riveted joint of equal static strength.

2. Spatter adjacent to the weld bead can cause a more serious stress raiser than the weld bead geometry and should be avoided.
3. Integral back-up strips should be avoided for fatigue applications.
4. The fatigue strengths of longitudinal welded butt joints are at least equal to those of transverse butt joints.
5. The use of unsymmetrical joints should be avoided for fatigue applications.
6. The effect of secondary flexing on the fatigue strength of unsymmetrical joints can be partially overcome by the use of stiffeners.
7. Butt-welded joints have higher fatigue strengths than fillet-welded joints.
8. Continuous fillet welds are superior to intermittent fillet welds for fatigue applications.
9. The fatigue strengths of plate specimens with fillet-welded attachments are about equal to those of butt-welded joints.

Residual Stresses Caused by Welding

Tests (15, 16) have shown that welding of aluminum alloys produces residual stresses whose maximum value is about equal to the minimum yield strength in the heat-affected zone. Figure 9 shows a typical distribution of residual stress parallel to the weld in a wide welded plate of 5456-H321. The dotted lines in the figure show how the residual stresses were reduced by cutting a 4-in. wide strip containing the weld out of the original 36-in. wide welded plate.

Tests of members under static loading, including column tests and falling-weight impact tests and falling-weight impact tests of plates, have shown that these residual welding stresses have no significant effect on structural behavior (15). Nor do these stresses introduce a susceptibility to stress corrosion cracking, according to results of tests on 5456-H321 plate (15).

Under conditions of cyclic loading, involving variations in applied stress from compression to tension, or from no stress to a tensile value, the residual tensile stresses in the weld region can have a significantly detrimental effect on fatigue strength at large numbers of cycles, as shown in Figure 10. Residual welding stresses can also

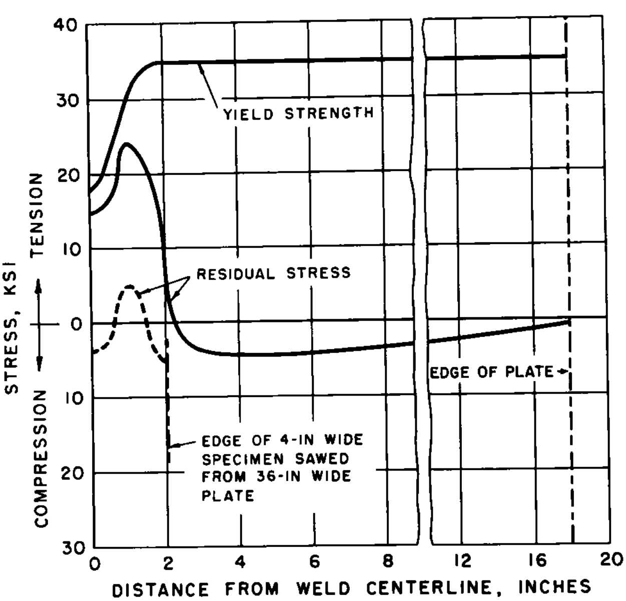


Figure 9. Distribution of yield strength and residual stresses in a 36-in. wide longitudinally welded plate, 5456-H321 plate, $\frac{1}{2}$ in. thick—5556 electrode.

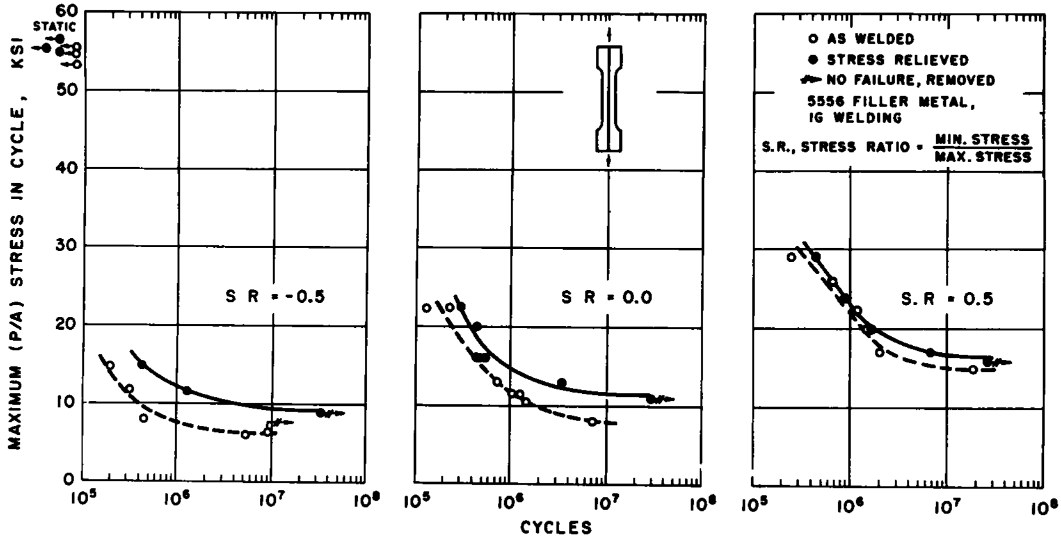


Figure 10. Effect of stress relief on fatigue strength of 5456-H321 longitudinal butt welds.

be undesirable if a weldment is to be machined after welding. Machining may disturb the residual stress balance, which will result in warping of the part.

Residual welding stresses in nonheat-treatable aluminum alloys can be reduced appreciably by a thermal treatment, with little sacrifice in strength. Tests on welded 5456-H321 plate indicate that holding weldments in this alloy at 525 F for about 15 min will reduce the residual welding stresses about 75 percent with no significant loss in tensile strength and less than 10 percent reduction in yield strength of the base material (15). The minimum yield strength in the heat affected zone may actually be increased slightly.

Design of Welded Tension Members

Tension members with transverse butt welds are designed on the basis of the minimum values of tensile strength and tensile yield strength across butt welds, which were discussed earlier. For highway structures, it is recommended that basic allowable tensile stresses be determined by applying a factor of safety of 2.2 to the minimum tensile strength and a factor of safety of 1.85 to the minimum tensile yield strength. The allowable stress is the lower of the two resulting values. As previously discussed, it is recommended that for weldments that receive only visual inspection the minimum tensile strength for design purposes be taken as 90 percent of the values given in Table 1

Frequently welds in tension members are so located that less than the entire cross-sections is affected by the heat of welding. In this case the allowable stress need not be as low as it would be if the entire cross-section were welded. Figure 11 shows the result of tension tests on specimens of ½-in. plate with longitudinal welds. As the width of the specimen was increased, so that a smaller portion of the cross-section was affected by the heat of welding, the experimental values of tensile and yield strength approached the strength of the unaffected parent metal. The curves in the figure represent strength values calculated from

$$f_{pw} = f_n - \frac{A_w}{A} (f_n - f_w) \tag{1}$$

in which

f_{pw} = strength* of member with only part of cross section affected by heat of welding.

f_n = strength* of unaffected parent metal.
f_w = strength* of material in reduced-strength sone.
A = area of cross-section.
A_w = area within area A that lies within reduced-strength zone.

The extent of the reduced-strength zone for the test specimens in Figure 11 was about 0.55 in. on either side of the weld, as determined by hardness surveys.

Eq. 1 is derived by assuming that the strength of the entire member is equal to the weighted average of the strength of the material in the reduced-strength zone and the strength of the unaffected parent metal outside this zone. Eq. 1 can be used for design purposes if the strength values are replaced by allowable stresses.

Design of Welded Compression Members

The most common type of welded compression member is a column supported at both ends with welds at the points of support. Tests and analyses have demonstrated that the welds have little effect on the column strength of such a member in the range of slenderness ratios where allowable stresses are controlled by column buckling rather than by yielding, provided that the ends are simply supported (16). Therefore, simply-supported compression members with welds at the ends can be designed on the basis of column formulas applicable to the unaffected parent metal, with a cut-off at an allowable stress based on the yield strength across butt welds. The strength of aluminum columns is given by the Euler column formula in the elastic stress range

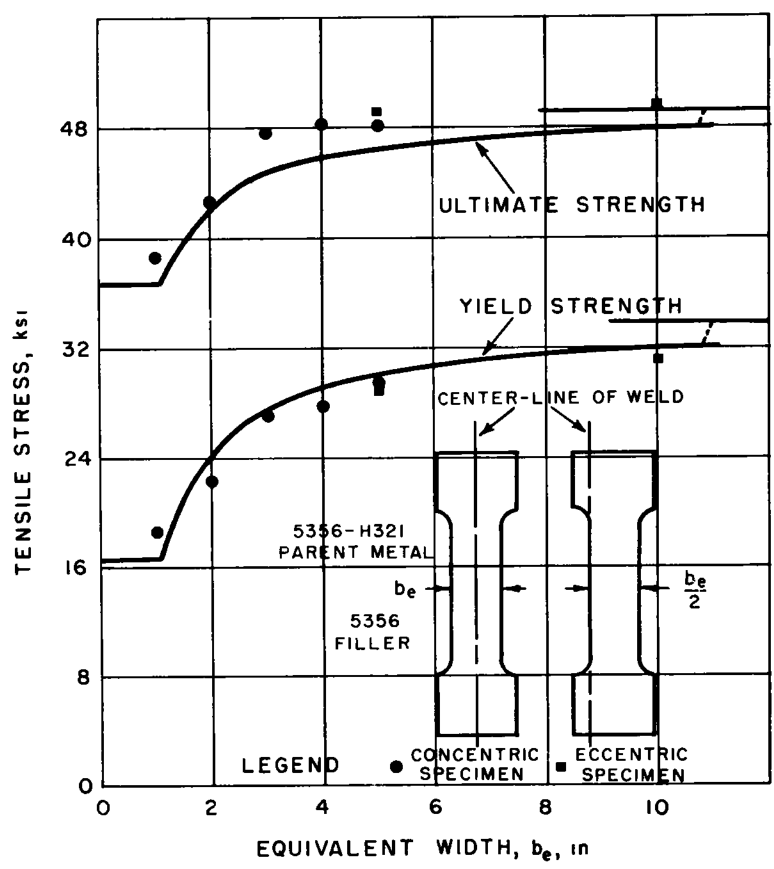


Figure 11. Results of tension tests of longitudinally welded members.

and a straight line approximation to the tangent modulus column formula in the inelastic stress range (17). Column formulas for the various commercial alloys are published in the Alcoa Structural Handbook (18).

If a column supported at both ends has welds at locations other than the ends (say, farther than 0.05L from the ends, where L is the distance between supports), the welding may have an appreciable effect on the column strength. The following equations may be applied to such members:

For $KL/r \le C_1$

$$f_c = 6.5\, f_y\,(1 - 0.04\sqrt{f_{yh}})\,/\,\sqrt{KL/r} \qquad (2)$$

For $KL/r > C_1$

$$f_c = \pi^2 E/(KL/r)^2 \qquad (3)$$

in which

f_c = column strength, ksi.
f_{yh} = compressive yield strength in reduced strength zone (considered to be equal to tensile yield strength in the reduced-strength zone), ksi.
K = end fixity coefficient, equal to 1.0 for columns simply supported at both ends and 2.0 for cantilever columns.
L = length of column, in.
r = least radius of gyration of column, in.
E = modulus of elasticity, ksi.

$$C_1 = \left\{\pi^2 E/[6.5\, f_{yh}\,(1 - 0.04\sqrt{f_{yh}})]\right\}^{2/3}$$

For columns with welds at locations other than the ends but with less than the full cross-section affected by the heat of welding, the column strength for design purposes can be determined by using Eq. 1 to interpolate between the allowable column strength for a member without welds and the column strength given by Eq. 2 for a member with the entire cross-section affected by the heat of welding.

In the case of cantilever columns, a weld at the end affects the column strength as much or more than a weld at any other location. Therefore, Eqs. 2 and 3 should be used to design such members, regardless of the weld location.

Local buckling of thin plate elements in the presence of welds can be calculated from the following equations:

For $k'b/t \le C_2$

$$f_{cr} = 7.5\, f_y\,(1 - 0.04\sqrt{f_{yh}})/\sqrt{k'b/t} \qquad (4)$$

For $k'b/t > C_2$

$$f_{cr} = \pi^2 E/(k'b/t)^2 \qquad (5)$$

in which

f_{cr} = local buckling stress.
b = width of plate element, in.
t = thickness of plate element, in.
k' = coefficient as given in Alcoa Structural Handbook (18, pp. 131-133).

$$C_2 = \left\{\pi^2 E\,[7.5\, f_{yh}\,(1 - 0.04\sqrt{f_{yh}})]\right\}^{2/3}$$

f_{yh} = and E as previously defined.

Local buckling of elements not affected by heat of welding can be calculated from the column formulas as indicated in the Alcoa Structural Handbook (18, pp. 130-137).

Allowable stresses for compression members are determined by applying the factor of safety on yielding (a value of 1.85 is recommended) to the compressive yield strength and the factor of safety on ultimate strength (a value of 2.20 is recommended) to the column-buckling strength and using the smaller of the two resulting allowable stresses.

Design of Welded Beams

Allowable tensile stresses in beams are affected by welding in the same way as allowable tensile stresses in other tension members. Similarly, allowable compressive stresses in beams are affected by welding in the same way as are allowable compressive stresses in columns. In other words, welds at the points of lateral support have little effect on the lateral buckling strength of a beam, although the lateral buckling strength may be reduced by welds at other locations. The lateral buckling strength of single web beams, either with or without welds, can be determined from the formulas for column strength by substituting the following value of equivalent slenderness ratio for KL/r in the column-buckling formulas:

$$\frac{KL}{r} = \frac{L_b}{1.2\, r_y} \qquad (6)$$

in which

L_b = length of beam between points of lateral support, in.

r_y = radius of gyration of beam about axis parallel to web (for a beam unsymmetrical about horizontal axis, r_y is calculated as though both flanges were the same as compression flange), in.

Use of the preceding values of equivalent slenderness ratio in the column formulas represents a conservative approximation, especially for values of L_b/r_y greater than about 50. For such beams, the designer may wish to compute a more precise value of equivalent slenderness ratio from the formulas in the Alcoa Structural Handbook (18, pp. 118-129).

Application of these principles to the design of beams in which the strength is controlled by yielding or fracture rather than by lateral buckling and comparisons with test results are illustrated elsewhere (19).

Design Specifications

The design principles discussed in this section were used as a basis for the allowable stresses in aluminum alloys quoted in recent specifications adopted by the American Association of State Highway Officials (20). These are the most up-to-date specifications for welded aluminum alloy structures that have been officially adopted by any code-writing body.

WELDING

Welded aluminum construction as presently used on American highway structures utilizes almost all of the commercial welding processes. This includes arc welding, resistance welding, gas welding, arc cutting, some brazing, and a little soldering. The following discussion, however, is limited to arc welding and, more specifically, to the inert gas welding processes.

Welded structures made from the ferrous metals are almost universally welded with a flux-coated metal electrode. Welding equipment, trained personnel, and procedure are widely available. It would be a tremendous advantage if aluminum alloy structures could also be welded with this process, but so far no generally acceptable coated welding electrode has appeared.

There seems to be little chance that coated electrode welding will displace the inert gas methods in the foreseeable future even though development work is continuing. The Norwegian marine industry, for instance, has developed an electrode for shipyard use, and several British and American electrodes are commercially available.

All of these electrodes are coated with fluxes made from chlorides and fluorides, and residual fluxes in the presence of moisture may attack the metal. Although this condition seldom affects the structural integrity of a part, it does produce an undesirable appearance. Painting a flux-contaminated surface results in blistered paint at the joints, and many cases it is not feasible to remove welding flux, particularly on field-welded structures.

The inert gas arc welding processes, performed with either consumable electrode (mig welding) or with tungsten arc (tig welding), are used almost universally for either field- or shop-welding aluminum structures. Both processes make superior joints from the standpoint of consistent weld soundness and contours, both can be used for welding in any position, and neither require post-weld cleaning. These processes do require special equipment and operator training, but such factors are no longer limitations because of the wide distribution of suitable equipment and the availability of training schools and other opportunities for self-training.

Tungsten Arc (Tig) Welding Equipment

Tig welding is most suitable for welding metal from 0.050 up to 0.5 in. thick. In most cases, welding is done with an alternating current generator or transformer that can be adjusted to the capacity needed to melt and weld a particular joint. A shield of argon or helium or a mixture of these gases flows through a gas cup positioned around a tungsten electrode.

An almost universally used appurtenance to provide an ionization path for the welding current is superimposed high frequency current on the secondary or welding arc circuit. This makes arc starting easier and re-establishes an arc that is extinguished accidentally.

Welding with direct current is widely used when welding the ferrous materials, but is seldom used in welding aluminum. An exception is automatic shop welding to make tube and pipe where DC with straight polarity is used.

Many publications, some of which are listed in the references show machine settings and other operating factors, but such data is not repeated here (21).

Tig welding has been used for a great deal of field welding, particularly on cross-country pipe line. Welds of excellent soundness are more easily achieved with this process. There is a substantial sacrifice in welding speed if this factor is measured in the pounds of metal that are laid down per unit of time when comparing speed of mig welding. This often is no disadvantage in field welding where the relatively slow rate of metal deposition makes it possible to get better control without exact joint fit or positioning the work. When welding decorative parts, tig welding will frequently make the best appearing joints.

Tungsten Arc DC Welding

Although most tig welding is done with AC, there is some use of DC either with direct or reverse polarity. These procedures deserve attention in view of their potential use for maintenance and repair and for welding the high-strength heat-treatable alloys.

Direct-current reverse polarity (electrode positive) can be used with almost any general purpose DC welding power supply and is characterized by shallow penetration, very easy arc-control, and exceptional "arc-cleaning." It has been found useful for repair operations on parts made from sheet and tubing. It is almost always used with argon gas because this provides the most easily controlled arc. In view of the wide availability of DC power sources, this method has the desirable quality of very low equipment cost.

Direct current straight polarity tig welding is used only for mechanized welding. The greatest welding heat is generated at the work surface, as contrasted to AC or reverse polarity. This results in a cool tungsten electrode and the ability to attain highest welding speeds because maximum current can be delivered to the joint. There is little or no cleaning action, and the arc must be operated at such short arc length as to require automatic torch positioning and traverse mechanisms.

The narrow weld zone is advantageous for welding the high-strength heat-treatable alloys. Best strength is obtained with a narrow cast weld and transition zone on such materials.

Consumable Electrode (Mig) Welding Equipment

Most structual welding on aluminim alloy is done on metal where the thickness is suitable for consumable electrode welding, and this process is preferred for most work. Metal from $\frac{1}{16}$ in. up to any desired thickness can be welded. Welded sections up to 2 in. thick are common, and metal up to 4 in. or more in thickness has been welded.

Filler metal is fed thru a jet of inert gas. Control of the arc length and rate of weld deposition is achieved by a high rate of electrode feed. Thus a high welding rate is inherent in the mig process.

There has been a multiplicity of types of equipment used for generating welding current, with some confusion on the part of fabricators as to what was best for their work. This situation is now welll stabilized on the use of direct current power source with reverse polarity for welding almost all aluminum. Generators delivering welding current with a drooping volt-ampere curve are widely available in industry. Such generators are driven by electric motors for shop work or by automotive engines for field work. This equipment is most economical in first cost, and is probably the most versatile in handling the multiplicity of geometries that must be welded in structural parts.

Constant arc voltage machines with control of slope to deliver either rising or drooping volt-ampere characteristics are finding wider use and, though more expensive, can be more universally used. The normal production variations such as joint fit, nonuniform section thickness, or varying surface conditions require somewhat less adjustment on the welders part with this type of apparatus. Electrode "stubbing" or "burn back" are more easily controllable.

Machine settings, joint design, and other welding variables are well-established for a wide range of work. This information is available elsewhere (21).

Arc Spot Welding

An adaption of the mig welding process to make spot welds has been developed. Joints are made by melting thru the top section into the lower part or along the edges of a piece making a localized fillet joint.

The process is useful for attaching stiffening members to sheet construction and for repair and maintenance operations. Equipment is portable so that either field or shop welding is feasible. In view of the low capital investment, the ability to weld when only one side is accessible, and the mobility of the equipment, the process should find increasing use.

Automatic timing equipment enables the operator to set welding time for the metal thickness concerned, and subsequent welding is then a matter of positioning the welding gun. Manual skill or a long training period is not required.

Best results are obtained with metal thicknesses below $\frac{1}{4}$ in. Joints are designed so that top member is about 25 percent thinner than the bottom part. This design is particularly desirable when the weld melts through on the face side and produces an undesirable appearance. Joints can be expected to have equal performance to electric resistance spot welds. Table 2 gives machine-setting data that are useful for design.

Choice of electrode alloy is the same as that normally recommended for the parent alloys used in the joint, although 4043 electrode finds wide use as a general purpose material.

Welding Characteristics of Structural Alloys

The first of this paper stated that alloys 6061-T6 and 6062-T6 are the most common structural alloys and that 5456 holds great promise for future development. It is not possible to express the weldability of these materials in single or even several criteria. Some important weldability factors are described here. A more detailed

TABLE 2

ARC SPOT WELDING CONTROL SETTING GUIDE[1]

Overlap Joints, .064 in. Sheet, 3/64 in. Diameter Electrode

Open Circuit Voltage: 32
Weld Time: Non-penetrating welds - 22 cycles
Penetrating welds - 30 cycles

Non-penetrating Welds	Wire Feed, in/min Filler Alloy			
Sheet Alloy	1100	4043	5554	5556
1100	475	400		550
3003	450	400		560
5154		400	440	475
6061		390	420	475
Penetrating Welds[2] All Alloys	550	500	550	600

1. Figures shown only for normally compatible sheet and filler alloy combinations. Each alloy listed representative of a family of alloys.

2. Against a copper backup plate.

analysis will depend on consideration of specific applications as there are perhaps as many aluminum alloys to fill specific needs as there are in the ferrous materials.

There are two factors in welding these alloys; namely, mechanical performance and weld soundness, that should be understood in using these materials. In alloys 6061 and 6062, mechanical strength is achieved by solution and precipitation heat treatment. Arc welding lowers the strength. This was described in the second section and the strength across a weld shown in Table 1. The minimum tensil strength of a butt weld in these alloys is 24,000 psi, chosen on the basis of many thousands of tests. This might be exceeded by judicious choice of welding practice to minimize the effect of welding heat. In practical use, however, such procedure control is seldom attempted.

Alloy 5456-H321 is a strain-hardened alloy which in the annealed or soft temper has a strength of 42,000 psi. This is also the strength in the "reduced strength zone" and consequently the minimum strength of a weld.

Another factor is ductility in the weld and the heat-affected zone. There are several ways to measure this factor so that one might choose a "free bend" test that measures in percent the elongation across a weld bead when it is bent until the metal fails. Joints in 6061-T6 or 6062-T6 show a ductility of 15 percent when welded with 4043 filler and 20 percent when welded with 5556 filler. Arc-welded joints in 5456 alloy have an elongation of 25 percent in this test.

Thus weldability, as measured by mechanical performance of a weld, static strength, and ductility, shows the superiority of 5456 alloy.

The alloy previously mentioned are representative of alloy groups each of which has parallel performance. The heat-treated alloy 6063 in a variety of tempers has a weld strength of 17,000 psi. Similarly, there are a series of nonheat-treatable alloys with lesser amounts of alloying constituents in which the minimum weld strength is the strength of the annealed temper, and the ductility of the joint is equal to or higher than that for welds in 5456. These alloys are generally called the 5000 series alloys and are usually chosen where the lower strength is adequate because the metal cost is lower

Weld soundness, the other major factor to be considered in judging weld quality, can be determined by detection of internal discontinuities in the weld or transition zone and of cracking when the weld cools. Internal discontinuities are defects in welds such as porosity, oxide film inclusions, contamination from other metals, poor fusion, or entrapped dirt. These conditions are the same for all aluminum alloys and are a function of welding technique. Correcting conditions that cause these defects cannot be accomplished by changing from a heat-treated alloy to a 5000 series alloy or vice versa.

Cracking, on the other hand, is a condition in which the alloy being welded has an important effect. During welding, cracks may occur in the deposited weld metal, in the transition zone, or at the end of a weld bead where a shrink or crater appears when the metal solidifies. The sensitivity to cracking in the weld zone has been measured quantitatively (23). The results can be summarized by stating that 5456 and other 5000 series alloys are much less sensitive to this condition that the heat treatable alloys.

This does not mean that welds in 6061 or 6062 are likely to have joint defects of this type. It does mean that more searching inspection is needed to control this condition, that weld procedure and tooling must be devised to put less restraint in the joints while cooling after welding. These precautions in general add to welding cost and thus may be undesirable.

Crater cracks at the end of a weld are controlled by welding technique and should not occur if the operator has been properly instructed.

There is general agreement in all specifications that cracks are not acceptable. In the aluminum alloys, elimination of cracks is accomplished by chipping or machining away the cracked area and then rewelding. Welding over a crack without removing it seldom eliminates the crack.

Qualification of Procedure and Operations

Welding codes and specifications for arc welding the aluminum alloys have been developed for a wide range of work, although no specific code for welding highway structures has been adopted. Probably the most widely used code, both in direct application, and as a model for new codes and specifications, is the Non-Ferrous Section IX "Welding" of the ASME Boiler and Pressure Vessel Code. The general system outlined in the code is much the same as that used on the ferrous materials.

In this code, a fabricator welds a test plate with the process and procedure he proposes to use on the structure, and the weld must meet minimum requirements in strength and soundness. In addition, each operator demonstrates his ability to make a weld, meeting minimum soundness requirements by welding a test plate. This is about all a welding code can accomplish in control of fabricating by welding. The requirement of code compliance before production is particularly useful for fabricators who are breaking into the field or are reactivating a department to weld aluminum structures.

Specific code requirements may be varied to suit particular problems. The following suggestions are offered for general use as an outline for code requirements. Details to meet specific jobs may then be developed.

Welding procedure is examined by making a butt joint in a metal thickness at least one-half that of the maximum thickness of the production parts with the welding process to be used in production. Strength is measured on two reduced section tensile specimens. Strength requirements are the minimum values shown in Table 1 for the alloy being welded.

Soundness is established by making face- and root-guided bend tests. Most aluminum alloy joints are bent to a 2T radius, except joints in 5456 alloy which are bent to a 3⅓T radius, and joints in 6061 which are bent to an 8T radius. Transverse guided bend tests are sometimes difficult to control because the parent metal, the heat affected zone and the cast weld zone have different bending properties. In this case, a longitudinal bend is made in which the specimen is made with the weld zone on the centerline and parallel to the sides.

Operator qualification may or may not be a requirement on the basis that inspection of the finished joints is essential to establish weld quality. Qualification does not insure good welds in the production parts.

In the event operator qualification is a requirement, it can be accomplished by welding a test plate to the same specifications as the procedure qualification plate. The major interest in the case of an operator is weld soundness; therefore (2), guided bend specimens are prepared and tested.

There is little difference in welding the various aluminum alloys discussed previously. This leads to the recommendation that operator qualification on any aluminum alloy is adequate for welding any of the other alloys in Table 1.

Filler Metal

The commonly used filler metals for inert gas arc welding are given in Table 1. These are available from distributors' stocks on short notice. The choice given here is suitable for most structural welding that is exposed to normal atmospheric service.

Occasionally, special exposure conditions are met with, and sometimes dissimilar alloy combinations must be welded. Table 3 gives a recommendation for six classifications of service requirements for welding an alloy to itself or to compatible alloys that might be used in combination.

Joint Preparation, Pre- and Post-Weld Cleaning

Joint preparation in welded structures is based on the amount of weld metal required to meet the strength and service conditions on the joint. Therefore, the aluminum alloys, the entire cross-section usually must be welded in butt joints, and a specified fillet size must be achieved in fillet joints.

In butt joints, the joint must be opened by a V, J, or U type of edge preparation to permit the weld to be made by melting to abutting edges and flowing in separate filler metal. In manual welding, and particularly in the fusion of a butt joint by heat alone, there is a greater likelihood of oxide film entrapment, cracking, or lack of complete fusion of the cross-section.

In welding aluminum alloys with the inert gas arc welding procedures, fusion to, but not beyond the root of a fillet weld, is attained. In a butt joint, fusion will occur not more than $\frac{1}{8}$ in. deeper than the root of the edge preparation. This is somewhat different than is experienced with the ferrous metals and, if understood, will lead to making sound joints that will meet the standard X-ray soundness requirements and have good static and fatigue strength.

Whether pre-cleaning of the aluminum surface is done before welding with the inert gas arc welding processes depends on the type of contamination present. The only two factors of importance in pre-cleaning are the natural oxide film and the presence of hydrocarbons such as oil, grease, or dirt.

Aluminum products as received from the mill have a very thin, uniform oxide film on the surface and, in the majority of cases, require no pre-cleaning to remove the film. Sometimes, however, there are cases where the metal has been stored outdoors under such conditions that water stain occurs, where several heat treatments or anneals have been applied to a part to form it, or where a part has required repair welding after having been in service. In such cases, pre-cleaning is easily done with a manual or rotary wire brush to clean the area in the joint and perhaps an inch or so to each side.

The other type of contamination, caused by oils, grease, or dirt, should be removed by solvent cleaning before welding. The inert gas arc welding processes are not very tolerant of such contamination. It is difficult to make sound welds if any hydrocarbons are present.

Post-weld cleaning is not essential to improve weld performance. The consumable electrode process leaves a dust deposit on the weld surface that is composed mostly of aluminum oxide. If the deposit is objectionable from the standpoint of appearance, it can be removed by a brushing or wiping operation. In most cases, however, it is left until the normal weathering washes it away.

TABLE 3

CHOICE OF FILLER ALLOY FOR WELDING HIGHWAY STRUCTURES

TO WELD THIS ↓	TO THIS ↓	EASE OF WELDING						① STRENGTH OF WELDED JOINT "AS WELDED"						② CORROSION RESISTANCE						⑤ SERVICE AT SUSTAINED TEMPERATURE						COLOR MATCH AFTER ANODIZING						③ DUCTILITY					
	FILLER ALLOYS	1100	4043	5254	5554	5356	5556	1100	4043	5254	5554	5356	5556	1100	4043	5254	5554	5356	5556	1100	4043	5254	5554	5356	5556	1100	4043	5254	5554	5356	5556	1100	4043	5254	5554	5356	5556
5456	1100																																				
	3003		A			A	A		B			A	A		B			A	A											A	A		C			A	B
	ALCLAD 3003		A			A	A		B			A	A		B			A	A											A	A		C			A	B
	3004		A			A	A		C			B	A		B			A	A											A	A		C			A	B
	ALCLAD 3004		A			A	A		C			B	A		B			A	A											A	A		C			A	B
	5005 - 5050		A			A	A		B			A	A		B			A	A											A	A		C			A	B
	5052					A	A					B	A					A	A											A	A					A	B
	5154			B	B	A	A			C	C	B	A			A	A	A	A									B	A	A	A			A	A	A	B
	5454				B	A	A				C	B	A				A	B	B										A	A	A				A	A	B
	5086 5356					A	A					B	A					A	A											A	A					A	B
	5083 5456					A	A						A					A	A											A	A					A	B
	6063 2EC ④		A	B	B	A	A		B	B	B	A	A		A	A	A	A	A									B	A	A	A		C	A	A	A	B
	6061 ④		A	B	B	A	A		C	C	C	B	A		A	A	A	A	A									B	A	A	A		C	A	A	A	B
	6062 ④		A	B	B	A	A		C	C	C	B	A		A	A	A	A	A									B	A	A	A		C	A	A	A	B
6061	1100		A			B	B		A			A	A		A						A									A	A		C			A	B
	3003		A			B	B		B			A	A		A						A									A	A		C			A	B
	ALCLAD 3003		A			B	B		B			A	A		A			B	B		A									A	A		C			A	B
	3004		A			B	B		C			B	A		A						A									A	A		C			A	B
	ALCLAD 3004		A			B	B		C			B	A		A			B	B		A									A	A		C			A	B
	5005 - 5050		A			B	B		B			A	A		A						A									A	A		C			A	B
	5052		A	C	C	B	B		C	C	C	B	A		A	B	B				A		A					A	B	A	B		C	A	A	A	B
	5154		A	C	C	B	B		C	C	C	B	A		A	B	B											A	B	A	B		C	A	A	A	B
	5454		A	C	C	B	B		C	C	C	B	A		B	B	A				A		A					B	A	A	A		C	A	A	A	B
	5086 5356		A	B	B	A	A		C	C	C	B	A		A	A	A	A	A									B	A	A	A		C	A	A	A	B
	5083 5456		A	B	B	A	A		C																			B	A	A	A		C	A	A	A	B
	6063 2EC ④		A	C	C	B	B		B																			B	A	A	A		C	A	A	A	B
	6061 ④		A	C	C	B	B		C																			B	B	A	B		C	A	A	A	B
	6062 ④		A	C	C	B	B		C																			B	A	A	A		C	A	A	A	B
																																	C	A	A	A	B
6062	1100		A			B	B		A			A	A		A						A									A	A		C			A	B
	3003		A			B	B		B			A	A		A						A									A	A		C			A	B
	ALCLAD 3003		A			B	B		B			A	A		A			B	B		A									A	A		C			A	B
	3004		A			B	B		C			B	A		A						A									A	A		C			A	B
	ALCLAD 3004		A			B	B		C			B	A		A			B	B		A									A	A		C			A	B
	5005 - 5050		A			B	B		B			A	A		A						A									A	A		C			A	B
	5052		A	C	C	B	B		C	C	C	B	A		A	B	B				A		A					B	A	A	A		C	A	A	A	B
	5154		A	C	C	B	B		C	C	C	B	A		A	B	B											B	A	A	A		C	A	A	A	B
	5454		A	C	C	B	B		C	C	C	B	A		B	B	A				A		A					B	A	A	A		C	A	A	A	B
	5086 5356		A	B	B	A	A		C	C	C	B	A		A	A	A	A	A									B	A	A	A		C	A	A	A	B
	5083 5456		A	B	B	A	A		C	C	C	B	A		A	A	A	A	A									B	A	A	A		C	A	A	A	B
	6063 2EC ④		A	C	C	B	B		B	B	B	A	A		A	B	B	C	C		A		B					B	A	A	A		C	A	A	A	B
	6061 ④		A	C	C	B	B		C	C	C	B	A		A	B	B	C	C		A		B					B	A	A	A		C	A	A	A	B
	6062 ④		A	C	C	B	B		C	C	C	B	A		A	B	B	C	C		A		B					B	A	A	A		C	A	A	A	B

1. RATING APPLIES PARTICULARLY TO FILLET WELDS. ALL FILLER ALLOYS RATED WILL DEVELOP PRESENTLY SPECIFIED MINIMUM STRENGTHS IN BUTT WELDS
2. RATING BASED ON CONTINUOUS OR ALTERNATE IMMERSION IN FRESH OR SALT WATER.
3. RATING BASED ON FREE BEND ELONGATION OF THE WELD
4. RATING DOES NOT COVER THESE ALLOYS WHEN HEAT-TREATED AFTER WELDING.
5. ABOVE 150° F.

REFERENCES

1. "Painting." Jour. Struc. Div., ASCE, 82:ST3, Paper 970 (May 1956).
2. Alcoa Research Laboratories, "Compatibility of Aluminum with Alkaline Building Products." Corrosion Mag. (Sept. 1957).
3. "Specifications for the Design and Construction of Structural Supports for Highway Signs." AASHO.
4. Hill, H.N., Clark, J.W., and Brungraber, R.J., "Design of Welded Aluminum Structures." Jour. Struc. Div. ASCE, 86:ST6, Paper 2528 (June 1960).
5. Hill, H.N., Clark, J.W., and Brungraber, R.J., "Design of Welded Aluminum Structures." Jour. Struc. Div., Proc., ASCE, p. 101. (June 1960).
6. "Qualifications Standard for Welding Procedures, Welders, and Welding Operators." ASME Boiler and Pressure Vessel Code, Sec. IX.
7. Nelson, F.G., Jr., and Howell, F.M., "The Strength and Ductility of Welds in Aluminum Alloy Plate." Welding Jour. Res. Suppl. (Sept. 1952).
8. Hoglund, G.O., "Welding the Aluminum Alloys." Paper, USAF-sponsored Heavy Press-Missile Conf., Worcester, Mass. (Nov. 1956).
9. "The Weldable Aluminum Alloys." Modern Metals (May 1959).
10. Hartmann, E.C., Holt, M., and Zamboky, A.N., "Static and Fatigue Tests of Arc Welded Aluminum Alloy 61S-T Plate." Welding Jour. Res. Suppl., p. 129 (March 1947).
11. Hartmann, E.C., Holt, M., and Eaton, I.D., "Fatigue Strength of Butt Joints in 3/8-In. Thick Aluminum Alloy Plates." Welding Jour. Res. Suppl., p. 21 (Jan. 1954).
12. Baron, F., and Larson, E.W., Jr., "Comparison of Bolted and Riveted Joints." Trans. ASCE, 120:1322 (1955).
13. Hechtman, R.A., Young, D.R., Chin, A.G., and Savikko, E.R., "Slip of Joints Under Static Loads." Trans. ASCE, 120:1335 (1955).
14. Munse, W.H., Wright, D.T., and Newmark, N.M., "Laboratory Tests of Bolted Joints." Trans. ASCE, 120:1299 (1955).
15. Hill, H.N., "Residual Welding Stresses in Aluminum Alloys." Metal Progress, p. 92 (Aug. 1961).
16. Brungraber, R.J., and Clark, J.W., "Strength of Welded Aluminum Columns." Jour. Struc. Div., Proc., ASCE, p. 33 (Aug. 1960).
17. Hill, H.N., and Clark, J.W., "Straight Line Column Formula for Aluminum Alloys." Alcoa Res. Labs., Res. Paper 12 (1955).
18. "Alcoa Structural Handbook." Aluminum Co. of America (1960).
19. Brungraber, R.J., "Strength of Welded Aluminum Box Beams." Welding Jour. Res. Suppl., p. 417-S (Oct. 1960).
20. Special Subcommittee on Structural Supports for Highway Signs, "Specifications for the Design and Construction of Structural Supports for Highway Signs." AASHO (1961).
21. "Welding Alcoa Aluminum." Aluminum Co. of America (1958).
22. "Welding Handbook." American Welding Soc., Sec. IV (1961).
23. Dowd, J.D., "Weld Cracking of the Aluminum Alloys." Welding Jour. Res. Suppl. (Oct. 1952).

Structural Considerations and Development Of Aluminum Alloy Culvert

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This paper evaluates and compares an appraisal of the structural load capacity of aluminum alloy culvert pipe with existing methods of analysis. Not only are theoretical considerations discussed, but the work is supported by extensive structural fill tests. A fill height table based on these considerations is included for use in design.

•THE INTRODUCTION of aluminum alloy culvert pipe into the highway construction field required that an appraisal of its structural load capacity be evaluated and compared with existing methods of analysis. To do this it was necessary to re-evaluate the several methods of theoretical analysis as they would apply to the characteristics of aluminum alloy culvert. Generally, there are two recognized approaches to the design of flexible metal culvert: the moment of deflection method proposed by Spangler (2), and the compression ring theory proposed by White (8). In general, the distinction between the two approaches occurs by assuming whether the pipe will fail by collapse or buckling or by failure of the joint seam in shear or bending. It was felt that both methods must be evaluated in aluminum culvert design to determine which method of failure might occur first under normal conditions.

In the evaluation of flexible culvert by Spangler it was noted that he was unable to substantiate performance of the culvert pipe in deflection with evaluation of bending stress. It was felt that this could be explained by a modification of the load distribution applied to the culvert pipe. If it could be demonstrated that the stresses developed conform as well as the deflection, a single solution would include axial reaction and a means of comparison with the compression ring theory.

Finally, concurrent with predicitions of load-carrying capacity, it was felt that a series of structural fill tests on aluminum culvert were necessary to demonstrate that actual test results would follow predicted results. Accordingly, a series of tests on culver in 24-, 36-, 48-, and 60-in. sizes were conducted by Kaiser Aluminum at Permanente, Calif., in the Spring of 1961. The prime purpose of these tests was to obtain general corroboration of structural performance of aluminum pipe over a series of sizes. Sections included examples of full circle, vertically elongated, and strutted culvert.

Using the results of the tests and analytical evaluation to confirm the results, a fill height table could be developed (Table 1). Its placement is at this point so that the results can be available for review before the supporting details are developed.

DESCRIPTION OF TEST

The site selected for the structural fill tests was at Permanente, Calif., on the eastern side of the Santa Cruz Mountains approximately 60 mi south of San Francisco. The native soil is a sandy loam which was determined to have an optimum density of 136 pcf. Moisture content of the soil during the test period was 10 to 15 percent. The material had a liquid limit of 26.6, a plastic limit of 10.8, and a plasticity index of 15.8. The specific area selected is shown in Figure 1 and was centered into a small natural canyon which was shaped to contour for placing the pipe. A large quantity of

TABLE 1

FILL HEIGHT RECOMMENDATIONS FOR ALUMINUM ALLOY-CORRUGATED CULVERT[a]

Culvert Diameter (in.)	Type of Shape	Minimum Recommended Cover (in.)	Maximum Recommended Fill Ht. for Gauges and Thicknesses (ft.)			
			16	14	12	10
			0.060	0.075	0.105	0.135
8	Full circle	6	50			
10	Full circle	7	40			
12	Full circle	8	35			
15	Full circle	8	32			
18	Full circle	8	26			
21	Full circle	9	25[b]			
24	Full circle	9	15[b]	24[b]		
	5% vertically elongated	8	16[b]	26		
30	Full circle	9		21[b]		
	5% vertically elongated	9		24		
36	Full circle	10			21[c]	
	5% vertically elongated	10			21[c]	
42	Full circle	12			16	
	5% vertically elongated	12			20	
	5% field-strutted	12			30	
48	Full circle	15			15	
	5% vertically elongated	15			18	
	5% field-strutted	15			30	
54	Full circle	15			15	
	5% vertically elongated	15			18	20
	5% field-strutted	15			25	30
60	Full circle	18				14
	5% vertically elongated	18				17
	5% field-strutted	18				25
66	Full circle	21				13
	5% vertically elongated	21				15
	5% field-strutted	21				25
72	Full circle	24				12
	5% field-strutted	24				20
78	5% field-strutted	24				16

[a]Loading = AASHO-H20 Highway, July 1961, shape = 2 2/3 by 1/2 in. Values are for 80- to 85-percent compaction.

[b]Based on use of 3/8-in. aluminum alloy rivets. If conventional 5/16-in. aluminum alloy rivets used, fill heights should be reduced to 21 ft for 21-0.060 in., 13 ft for 24-0.060 in., 21 ft for 24-0.075 in., and 19 ft for 30-0.075 in.

[c]Based on use of 1/2-in. aluminum alloy rivets. If conventional 3/8-in. aluminum alloy rivets used, fill heights should be reduced to 18 ft.

borrowed material from an immediately adjacent hillside was available for the backfill operation and is located on the left side of Figure 1. The test was scheduled to commence in early April shortly after the winter rains had subsided. In this particular year the rains lasted somewhat longer so the actual placement of the material did not commence until April 24, 1961. The test period lasted approximately six weeks during which time no measurable rainfall was recorded. Average daily temperatures were approximately 70 F.

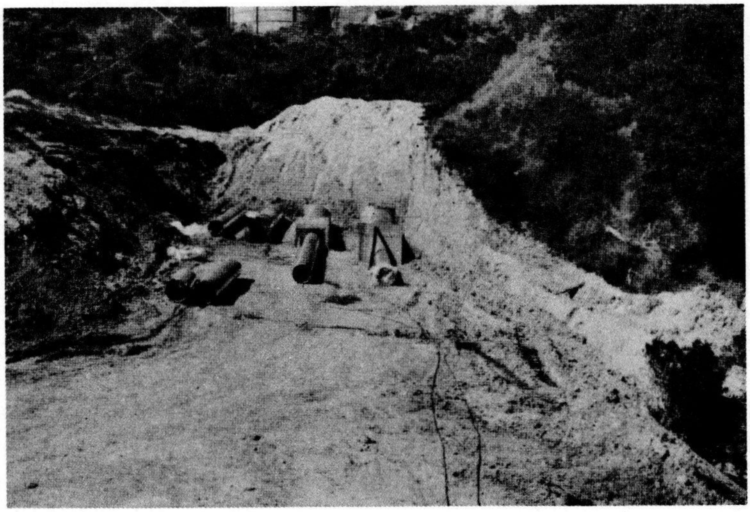


Figure 1. General view of test site showing pipe runs in approximate position.

The test site is not be used for traffic but can and will be left intact so that long-term readings can be taken to determine what consolidation effects develop. A drawing of the average sectional profile is shown as Figure 2.

DEEP FILL SECTIONS

Four series of culvert pipes of the conventional 2⅔-in. pitch by ½-in. depth corrugation were fabricated by three independent culvert manufacturers in California using standard fabrication practice for inclusion in these tests. The test runs consisted of 24-, 36-, 48-, and 60-in. pipe running parallel and spaced a sufficient distance apart to theoretically satisfy the case of positive projection loading conditions and minimize effect of one pipe on another. Each run of pipe consisted of a series of 10-ft long test sections selected and fabricated to demonstrate a series of engineering conditions from which information could be developed. A drawing has been prepared showing the arrangement of these sections (Fig. 2). In addition, Figure 3 shows the four parallel runs before completion of the installation of access tunnels and backfilling with each test section identified. The original surface below the pipe was tapered and leveled with approximately 1 ft of clean sand so that all four runs of pipe could be bedded with the tops at a common elevation.

The 24-in. run had four test sections: 16 gauge vertically elongated riveted, 16-gauge (0.060-in. thickness), 14-gauge (0.075-in. thickness) full circle riveted, and a 16-gauge full circle piece which was spot welded with a Sciaky three-phase spot welding machine. The vertically elongated piece was separated from the full circle pieces by a 4-ft preformed transition section and two couplings. This prevented carryover of structural effect from one shape to the other.

The 36-in. run had a section of 12-gauge (0.105-in. thickness) full circle riveted, a section of 12-gauge full circle with two spot welds per pitch, and a section of 12-gauge vertically elongated riveted pipe. For the 36-in. and larger pipes, ¾-in. plywood was used to block the ends with sufficient timber support to prevent the plywood from moving out of position during the test. At the downstream end a 24-in. hole was cut into the plywood through which a 24-in. culvert access tunnel was installed. This method of construction insured that the test sections would be under the maximum fill and that access to the pipe could still be maintained.

The 48-in. run had a 12-gauge full circle double-riveted section, a 12-gauge vertically elongated double-riveted section, and a 12-gauge timber strutted section using 4- by 4-in. timber struts against 4- by 4-in. sole and top runners with the struts placed on approximately 5-ft centers.

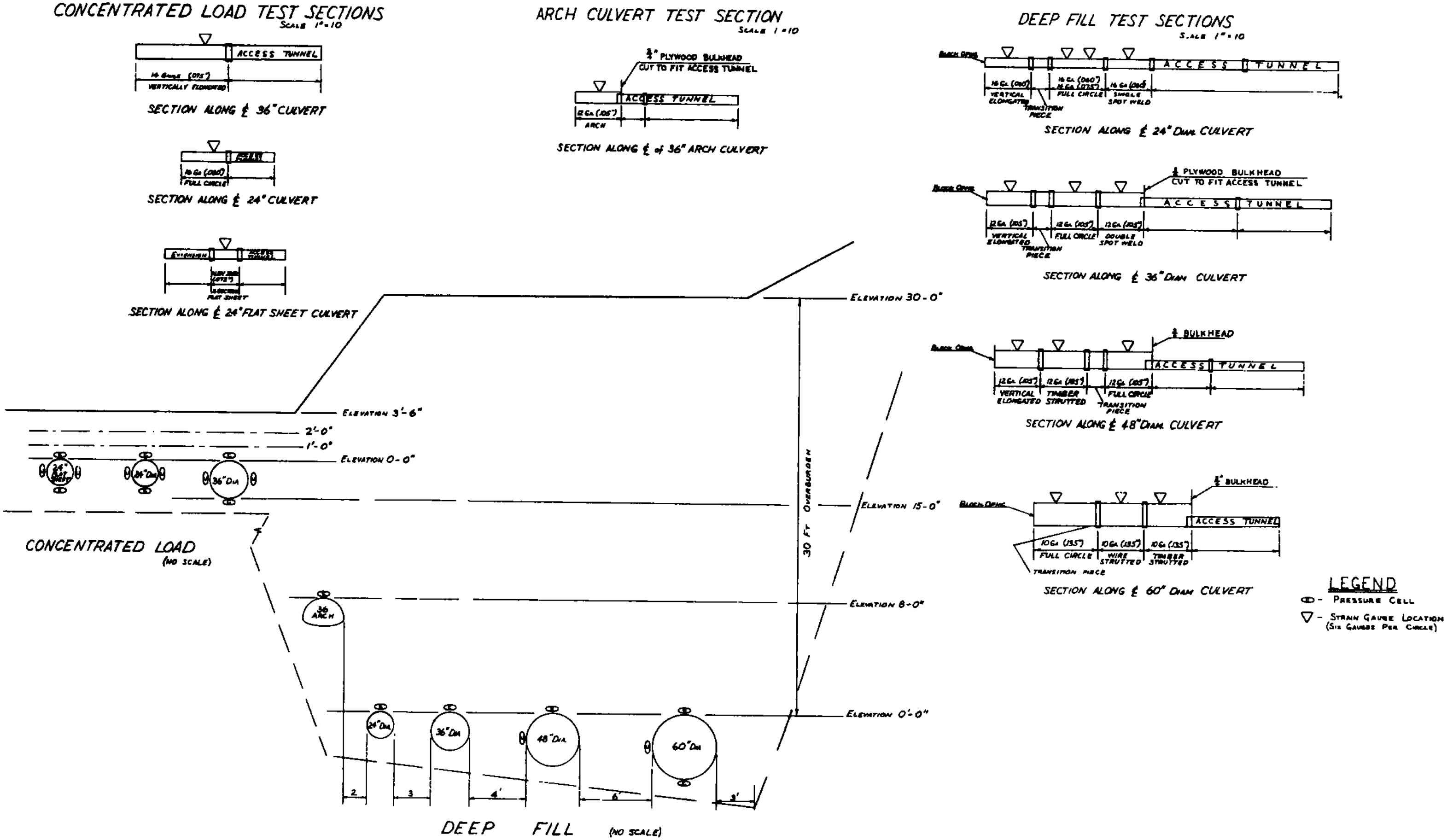


Figure 2. General arrangement, aluminum culvert tests, concentrated load and deep fill.

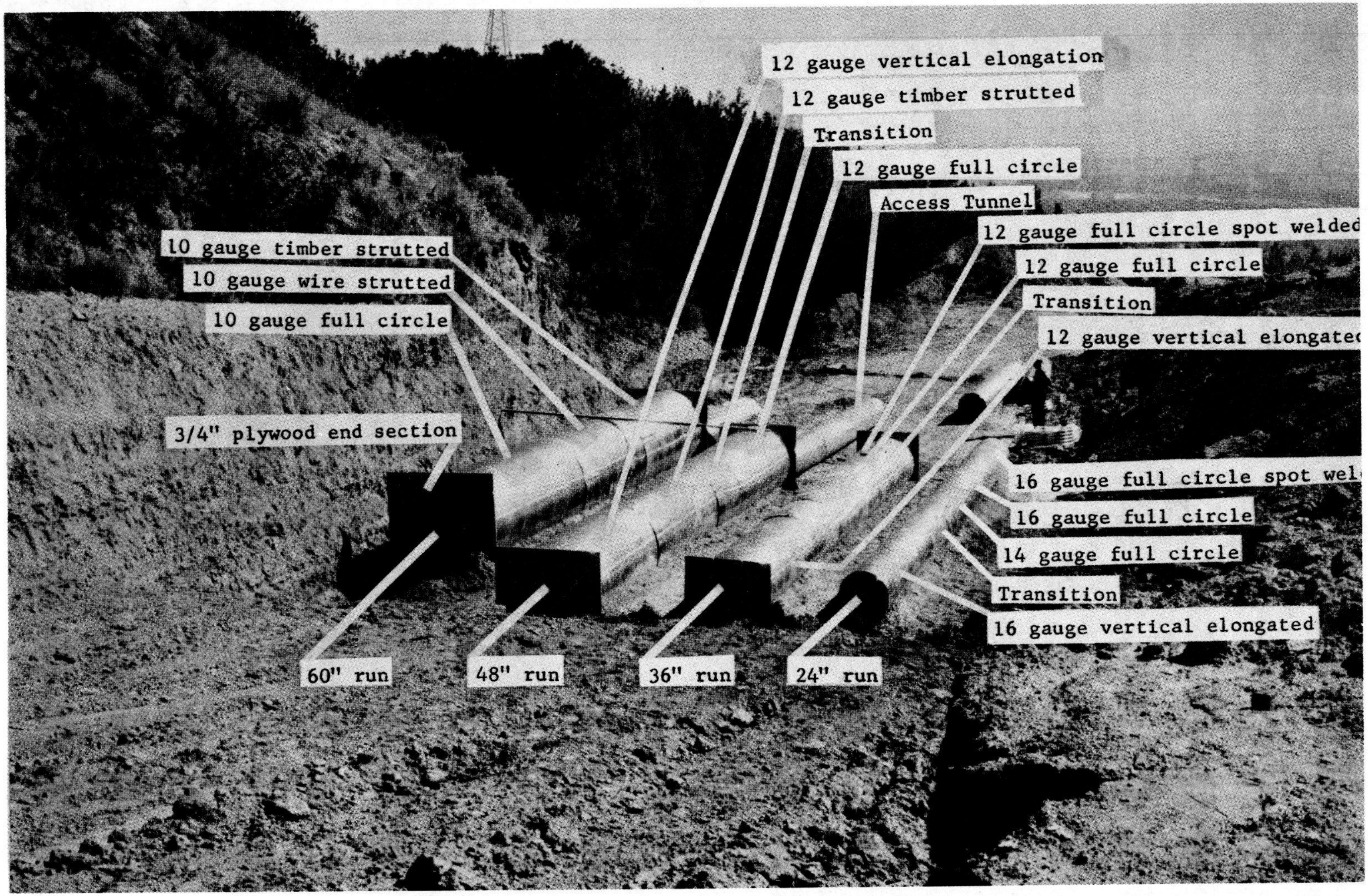


Figure 3. General arrangement of deep fill pipe showing location of test sections.

The 60-in. run had a section of 10-gauge (0.135-in. thickness) full circle double-riveted, a section of 10-gauge timber strutted with timber strutting following the procedure used in installation of the 48-in. section, and a section of 10-gauge wire strutted double-riveted pipe. The wire strutting was accomplished in the factory by threading $^3/_{16}$-in. wire loops across the centerline of the pipe at 2-ft intervals and twisting and upsetting the pipe until a 5 percent pre-set elongation was obtained. The transition in this case was done by making the full circle section 14 ft long and deforming the end.

Vertically elongated pipe in the 24-, 36-, and 48-in. sizes was elongated to shape in the factory without stays or other prestressing means. Timber strutting and wire strutting prestressed the pipe by elongating the vertical axis. Vertical elongation in all cases was 5 percent of the diameter of the pipe.

A test section of 36-in. 12-gauge arch pipe was installed at an elevation of approximately 8 ft above and to one side of the deep fill runs so that the structural characteristics of arching can be evaluated. It is located within the indentation in the soil over the 24-in. pipe of the deep fill in Figure 4.

Concentration Load Sections

So that means of investigation of performance of aluminum pipe under minimum fill or concentration load conditions could be investigated, three test sections were bedded into the test site above and to the left of the deep fill sections. These sections were 24-in. flat sheet, 24-in. corrugated culvert, and 36-in. corrugated culvert. The shape of the hill was such that the concentration load pipe could be bedded on 1 ft of clean sand over native, undisturbed material. The 24-in. flat sheet was 14 gauge and riveted, the 24-in. diameter culvert was 16 gauge corrugated and riveted, and the 36-in. diameter was 14 gauge corrugated and riveted. These pipes were placed parallel so that the top centers would be at the same elevation in a manner similar to the deep fill. Figure 4 shows the position of the three test sections above the deep fill pipe when 15 ft of fill was placed.

Instrumentation

In conducting a series of tests of this magnitude, it is highly desirable to provide as much instrumentation as possible which can be used to develop basic data. The variables affecting the performance of the culvert were recognized to be soil load and distribution, pipe deflection, and pipe stresses. Accordingly, a series of pressure cells, deflection indicators, and strain gauges were installed on the various test sections to serve as the instrumentation for this test. In addition, the Pittsburgh Testing Laboratory was retained to determine compaction curves and compaction values of the soil and to serve as supervisor of instrumentation for this test.

Pressure cells used in this test were made by using an incompressible fluid (i.e., water) to transmit pressure in a system consisting of a common hot water bottle as the pressure-sensing bladder, to which was attached $^1/_4$-in. diameter aluminum tubing and an indicating pressure gauge. Pressure cells were placed at selected locations at the invert, sides, and across the tops of the deep fill and concentration load tests. Care was taken to level the cells by installing the gauges at the same elevation as the bags. The cells across the top of the pipe were used to demonstrate pressure actually applied to the pipe, and hence, loading actually applied to the pipe. Pressure cells placed against the side of the culvert were used to measure the modulus of soil reaction developed during the loading process on the culvert. Pressure cells at the invert were installed to give some indication of the pressure transmission through the structure. The specific location of these pressure cells is indicated on the master arrangment drawing (Fig. 2) for the culvert installation.

To determine accurately total and incremental deflections for zero to maximum fill conditions, deflection indicators were fabricated and placed in selected culvert sections. These were made up from $^3/_4$-in. wood dowels which telescoped into $^3/_4$-in. I. D. aluminum conduit and were spring loaded to hold position. A short section of conduit was screwed to a section of dowl and washers and spring were inserted inside the conduit against the dowel. Another portion of dowl was inserted into the cavity. An aluminum plate was welded to the conduit with scaled readings in $^1/_4$-in. increments with an indicator attached to the floating dowel. These indicators were subsequently read by field glasses and spotlight from inside the access tunnel. In addition to the above, initial and final deflections under fill were measured for all test sections not equipped with indicators.

A series of SR-4 foil strain gauges were attached to the quadrant points for each test section. A total of six gauges was selected for each group; a pair at the top, a pair at one side, one at the invert, and one in the opposite side. The paired gauges were placed on the inside crest and inside crest and inside valley so that bending could be indicated. The gauges were installed by first sanding smooth and thoroughly cleaning the aluminum surface with acetone, after which the strain gauges were attached using Duco cement. To insure proper linearity of the gauges, a template was used which conformed to the contour of the culvert and accurately indicated the center of the valley and the ridge. Proper pressures were applied to the gauges during the curing stage by placing pieces of foam rubber over the gauges and strutting between the walls of the culvert. No. 20 wire was then soldered to the gauge leads and initial tests for grounding were made at this point. The outside surfaces of the culvert at the gauge locations were then warmed up and Petrocene wax was applied over the gauges to a thickness of approximately $^3/_{16}$ in. in an effort to prevent water intrusion. A final ground check was then taken. All six gauges for each station were collected into a harness wired to a common six-pole selector switch which in turn was wired to a 20-channel switching instrument. Three compensating gauges were used. These were adhered to pieces of aluminum plate, waterproofed, and taped to the inside wall of the culvert in the gauge location areas.

Where gauges were paired, it was deemed advisable to install one gauge on the inside ridge and the opposite gauge on the inside of the valley, thus preventing any contact

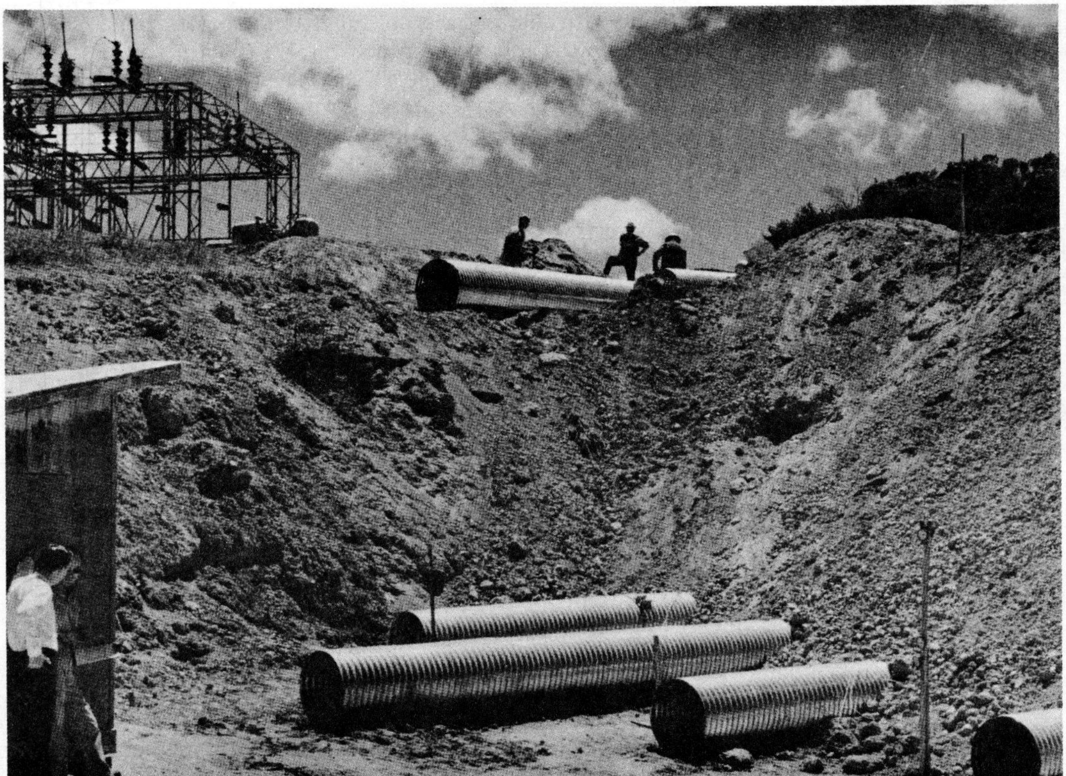


Figure 4. General view from below showing 24-, 36-, 48-, and 60-in. runs across the bottom, the arch pipe above 24-in. one, and the three concentration load pipes at top; fill over deep pipe was 15 ft at this point.

with the soil. It is necessary then to extend the reading of the valley gauge to the extreme fiber by applying an algebraic proportion. Once this is done, both the average compressive stress across the section, which is an indication of the shear strength across the joint, and the maximum bending stress could be determined for each pair.

Backfill

Each of the test sections were carefully bedded into a shaped sand base with the bedding approximately 12- to 15-in. wide and extending the full length. The initial backfilling of the deep fill sections was with a D4 with a loader attachment. The D4 was selected because of its maneuverability in the bottom of the canyon. A loader attachment was desirable because of the relatively long distance that material had to be moved to be placed to backfill properly the pipe to the top. To accomplish the backfill properly, placement of material commenced in one corner and radiated to the side and forward from that point as in Figure 5. Proper compaction was developed by spreading the material during placement and working with pneumatically operated tampers. This procedure was continued in approximately 1-ft lifts until all pipes were completely covered. Compaction tests in this area showed the average Proctor density to be 83 percent.

One of the purposes of this test was to establish the performance of aluminum pipe under what is accepted to be normal or standard backfill conditions. It is recognized that many specifications require compactions of 95 percent in backfilling operation, but it is also a common fact that the average installation does not secure optimum compaction but merely secures "good" compaction. The structural fill tests were designed to follow the latter pattern and from them it was possible to conclude that normal compaction around culvert pipe would be of the order of 80 to 85 percent Proctor density. This practice and theory was followed throughout the entire fill operation.

Once the material was compacted to the top of the pipe, the D4 loader was dismissed and a D7 loader brought onto the job in its place. The D7 loader served as the means of placement and compaction throughout the remainder of the test. The average compaction attained under the track of the D7 loader by compacting lifts of approximately 8-in. thickness was 80 percent. Backfill was increased to progressive compacted fill heights in 1-ft increments through 6 ft, in 2-ft increments through 12 ft, and fills of 15, 20, 22, 26, and 30 ft.

When the fill was 5 ft over the deep fill pipe, both the timber and the wire struts were removed so that data could be obtained on the performance of pipe that had been previously elongated under the application of increasing loads instead of the common practice of leaving the struts in place until the fill was completed. During the removal of the struts, damage was sustained on the strain gauges for the 60-in. timber-strutted section destroying the apparent validity of stress results obtained from this section.

Pressure, strain, and deflection readings were recorded for each fill height increment for each section in each run.

The concentration load tests were developed in an equal manner to the deep fill tests except that fill heights of 1, 2, and 3.5 ft were the only values considered. At each fill level a series of strain, pressure, and deflection readings were taken which would serve as the basis for analysis of data. Concentrated loads were developed by running one track of the D7 loader onto a series of ¾-in. thick steel plates with 4- by 4-in. blocking above the plates to insure the total load was impressed on the plates. The plate sizes were 15, 18, and 24 in. square. Thus, a total of 12 complete sets of readings were taken.

LOADING ANALYSIS

There are several methods of estimating the load actually applied to the culvert pipe in this evaluation. A series of pressure cells were placed on the top of each pipe run to attempt to measure the actual pressure transmitted at the top of the pipe. The pressure exerted by the pipe at the center on the soil was also measured to assess the modulus of passive resistance. Attempts were made to observe and estimate the settlement

Figure 5. (a) Initial spreading of backfill with D4 loader. (b) progressive backfilling around deep pipe with pneumatic compacting; and (c) leveling backfill over deep fill with 1-ft fill (pipes in foreground are access tunnels).

of the pipe and the soil adjacent to the pipe so that the settlement ratio of the installation could be predicted.

As indicated earlier, the pipe was installed as a positive projection, the most severe design loading condition. However, the tests demonstrated that the installation had a high negative settlement ratio, and the solution in this case would approach the ditch condition. The actual load on the pipe would be considerably below the predicted load from the dead weight of the fill if this were the case. This condition did, in fact, occur and the computations must reflect this.

The pressure values are plotted in Figure 6. The pressure transmitted to the center of the pipe was shown to be considerably below the normal average pressure across the section after 5 ft of fill was reached. The probable pressure distribution at the top of the pipe is shown in Figure 7. A theoretical uniform pressure equivalent is also shown on the curve for purposes of evaluation using existing theories. It is probable that actual pressure distribution is trapezoidal in nature following the pipe deflection

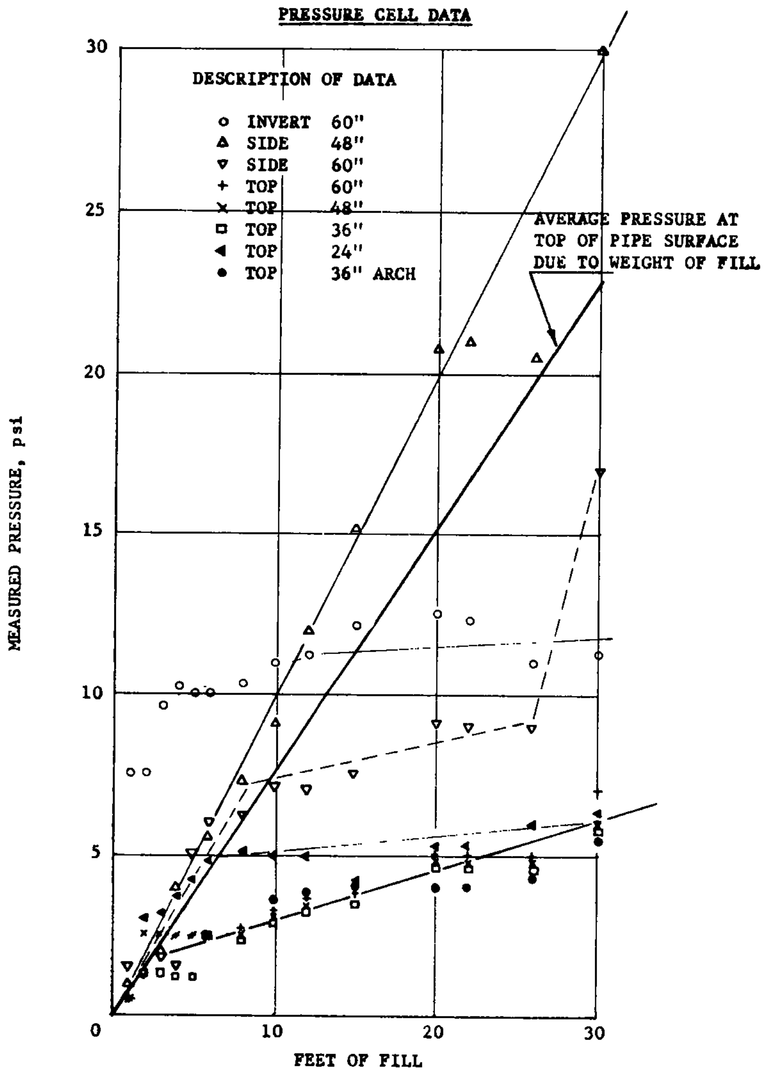


Figure 6. Pressure cell data.

values, but it is felt that a mean average uniform pressure will produce acceptable results in this case. This mean pressure will be used in computation of load factors following and is given in Table 2, Columns 3, 4, and 5.

The proper evaluation of flexible culvert by means of the Spangler (2) method depends on an accurate appraisal of the product of the settlement and projection ratio. Although the projection ratio, the ratio of distance from undisturbed surface of soil to top of pipe to pipe diameter, in this case could be accurately developed, the settlement ratio can only be estimated. Three comparisons can be developed to check the settlement ratio: pressure ratio, soil compaction deflection and pipe deflection, and visual observation of behavior of soil and pipe.

Moving now to the analysis of settlement ratio, it is necessary to evaluate first the derivation of the settlement ratio with Figure 8 for description.

$$r_s = \frac{(S_M + S_g) - (\Delta y + S_f)}{S_M} \tag{1}$$

in which

TABLE 2

PRESSURE ACROSS PIPE AND PIPE LOAD FACTORS FROM PRESSURE[a]

Culvert Diameter (in.)	Fill Height H (ft)	Pressure (psi) P_1	P_2	P Mean	Load W (lb/ft)	C_c from Pressure Data	$\frac{H}{B_c}$	$r_{s}p$ from Pressure Data
24	10	7.6	5	6.3	1,810	4.15	5	-0.10
	20	15.2	5.5	10.3	2,960	6.78	10	-0.35
	30	22.8	6.0	14.4	4,150	9.50	15	-0.40
36	10	7.6	3.3	5.5	2,380	2.42	3.3	-0.50
	20	15.2	4.6	9.9	4,280	4.36	6.7	-0.50
	30	22.8	6.0	14.4	6,220	6.34	10	-0.50
48	10	7.6	3.3	5.5	3,160	1.81	2.5	-0.50
	20	15.2	4.6	9.9	5,700	3.26	5.0	-0.50
	30	22.8	6.0	14.4	8,290	4.75	7.5	-0.50
60	10	7.6	3.3	5.5	3,960	1.46	2	-0.50
	20	15.2	4.6	9.9	7,130	2.61	4	-0.50
	30	22.8	6.0	14.4	10,360	3.80	6	-0.50

[a] P_1 = average pressure across top of pipe due to weight of fill (psi), P_2 = measured pressure at top center of pipe (psi). $P_{mean} = (P_1 + P_2)/z$, W 12 PB_c, B_c = pipe diameter in feet, and $C_c = W/wB_c^2$.

r_s = settlement ratio,
S_M = settlement of bottom critical plane,
S_g = settlement of bottom critical plane below soil,
Δy = deflection of top of pipe, and
S_f = settlement of bottom pipe.

This settlement ratio, when combined with the projection ratio, is the means by which which the load on the pipe can be determined. The computed load is expressed from

$$W = C_c w B_c^2 \qquad (2)$$

in which

W = load on pipe (lb per ft of length) (see eq. 1);
C_c = load coefficient, a function of fill height, settlement ratio, and projection ratio;
w = density of soil (pcf); and
B_c = width or diameter of pipe (ft)

Product of C_cB_c equals H if the settlement-projection ratio is zero; the full vertical wedge of the soil acts downward and $W = wHB_c$. C_c can be computed by know-

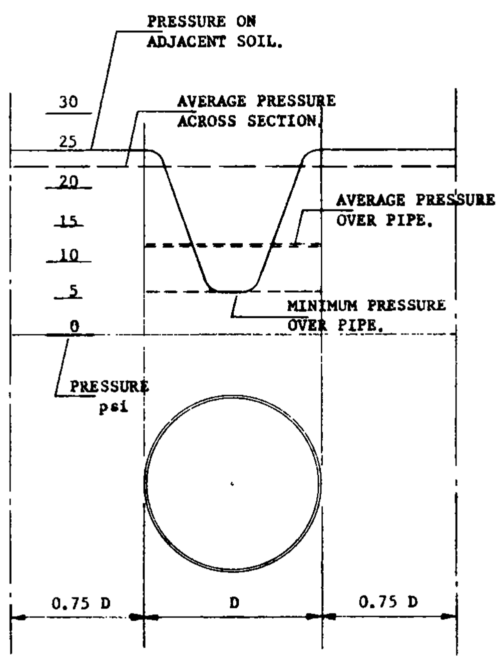


Figure 7. Pressure distribution across pipe plane under 30 ft of fill.

$$\text{SETTLEMENT RATIO} = r_s = \frac{(S_M + S_g) - (\Delta y + S_f)}{S_M}$$

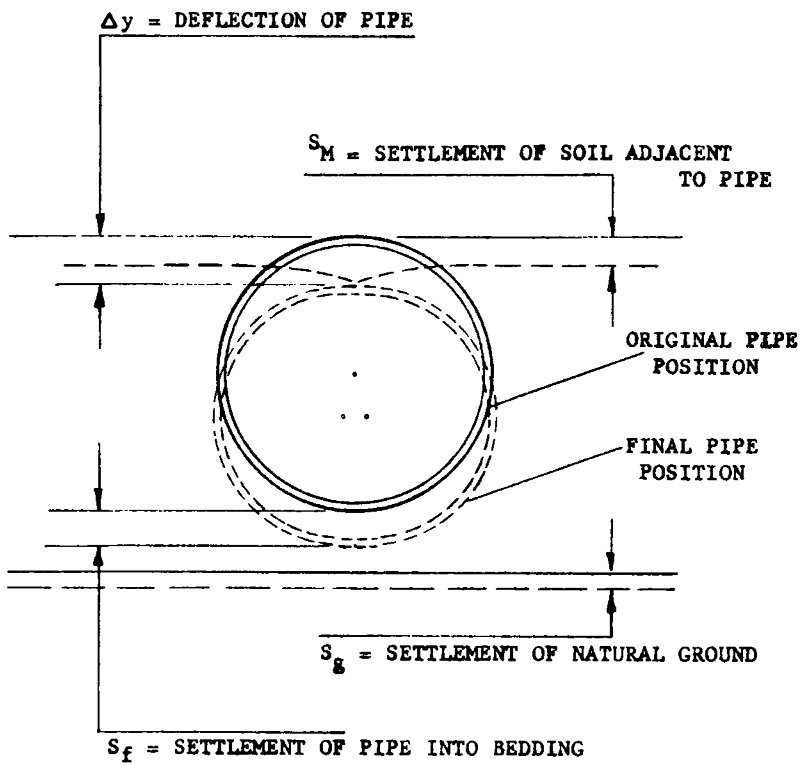


Figure 8. Settlement ratio for flexible pipe.

ing the average pressure on the soil above the pipe and comparing this with the density of soil and diameter. This is done in Table 2 and forms the basis for computations of load. This method serves also as a means for determination of the actual settlement ratio.

Observation showed the values of S_g and S_f, the settlement into the ground, to be very small and these values can be neglected in this test. Δy (which is equal to Δx) could be readily measured and is given in Table 3 as measured Δx. This leaves the evaluation of S_M necessary to complete the settlement ratio. Observation showed the soil to compress only a portion of that of the pipe. Thus, in the limit if S_M were zero, the settlement ratio would be negative infinite or equal in magnitude to the complete ditch condition. This confirms the trend shown in the pressure data to an approximate degree.

Marston and Spangler (3) were able to evaluate C_c into a family of curves relating the settlement-projection ratio and the ratio of fill height to pipe diameter. The computed values of C_c and r_{sp} taken from the pressure data are given in Table 2.

As an additional method of determining the settlement ratio, a series of laboratory compaction tests were conducted on 6-in. diameter by 6-in. deep soil specimens. These specimens were initially compacted to 74, 83, and 93 percent Proctor density and then subjected to up to 40-psi compressive stress. These results were then converted to percent consolidation (Fig. 9). Because the average compaction around the pipe was 83 percent, these data would indicate the percent settlement of the soil (S_M/D) to be 0.8 at 30 ft of fill. If the deflection of pipe only were considered, the computed settlement ratio would be approximately -5.0. Although the value determined in this simple means is substantially over the value computed from pressure data, the deviation in the load coefficient within this test is quite small. Considering the extreme difficulty in comparing this cylinder compaction test with field performance, which is of larger scale and is three dimensional, it is felt that reasonable correlation of experimental data was obtained. Certainly the true soil settlement would be higher than the laboratory value making the check between pressure determination and settlement determination in this test quite good.

SUPPORTING STRENGTH ANALYSIS

Before undertaking a discussion of the results of the structural fill tests, further analysis of loading and support strength should be undertaken. The method that considers the bending stresses of the culvert pipe with inclusion of soil resistance is the method of rational moments and deflection as proposed by Spangler (2). In this approach a loading pattern on the flexible pipe is developed which consists of a uniform pressure across the full top of the culvert, a parabolic-shaped soil resistance at the centerline, and a uniform but shorter width bearing pressure across the invert. This pressure distribution is shown as Figure 10. In this method of evaluation it is assumed that the pipe sustains a high deflection to ultimate failure before the average compressive stress on the sheet is sufficient to separate the riveted joints. The form of the equation proposed by Spangler is

$$\Delta x = \frac{D_L K W_c r^3}{EI + 0.061\, E' r^3} \tag{3}$$

in which

- Δx = horizontal deflection (in.) (Δx is equal to Δy, the vertical deflection);
- D_L = deflection lag factor, based on observed continuing deflection as soil completes its consolidation;
- K = bedding constant;
- W_c = soil load on pipe (lb per in.);
- r = mean pipe radius (in.);
- E = modulus of elasticity of pipe metal (psi);
- I = moment of inertia of pipe section (in.^{4} per in.);
- E' = modulus of soil reaction (psi), a measure of resistance of soil to horizontal expansion under load, equal to er;
- e = modulus of passive pressure of soil (psi/in.).

Significantly, this equation takes into account the structural effect of both the metal comprising the pipe and the soil resisting the expansion of the pipe at the centerline. Soil analysis will develop the load applied, bedding constant, and modulus of passive resistance of the soil. The remainder of the items are geometry of the structure.

In the tests conducted, W_c, K, and E' are values determined by performance of the soil; E, I, and r are determined by the geometry of the pipe itself; and Δx is the measured or predicted deflection of the pipe. D_L, the deflection lag factor, is unity when taken on the immediate completion of the test, increasing to a suggested 1.50 for design purposes when full consolidation of a typical soil has been completed.

The initial step in evaluation of the data is to calculate deflection from actual test results using Eq. 3 and compare it with the measured deflection. After corroboration of data, the second step is to develop design fill heights using a "standard" set of soil conditions.

TABLE 3

MODULUS OF PASSIVE RESISTANCE OF SOIL[a]

Fill (ft)	48-In. Culvert P Pressure (psi)	$\Delta \frac{x}{2}$	e (psi/in.)	60-In. Culvert P Pressure (psi)	$\Delta \frac{x}{2}$	e (psi/in.)
5	5.0	0.25	20.0	5.0	0.12	41.6
10	9.1	0.34	26.7	7.1	0.25	28.4
15	15.2	0.56	27.2	7.5	0.34	22.0
20	20.8	0.81	25.7	9.1	0.50	18.2
25	20.5	1.31	15.6	9.0	0.75	12.0
30				17.1	1.37	12.5
Average e =			23.0		=	22.4
E' = er =			558		=	678

[a]A typical value of E' of 600 psi is used in checking results.

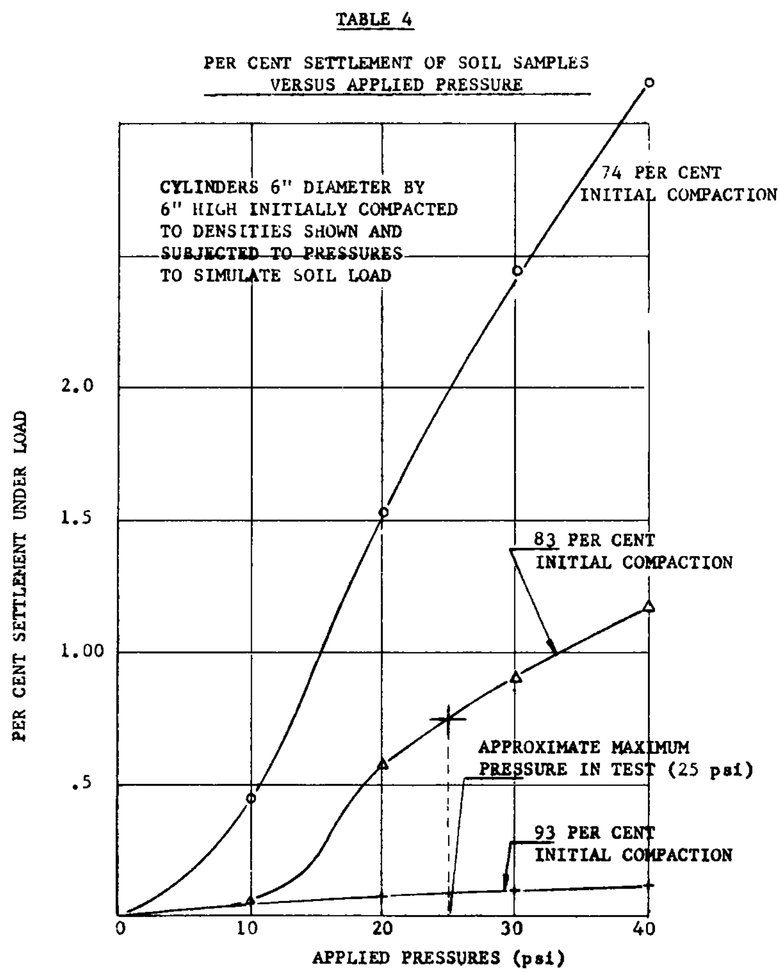


Figure 9. Percent settlement of soil samples vs applied pressure.

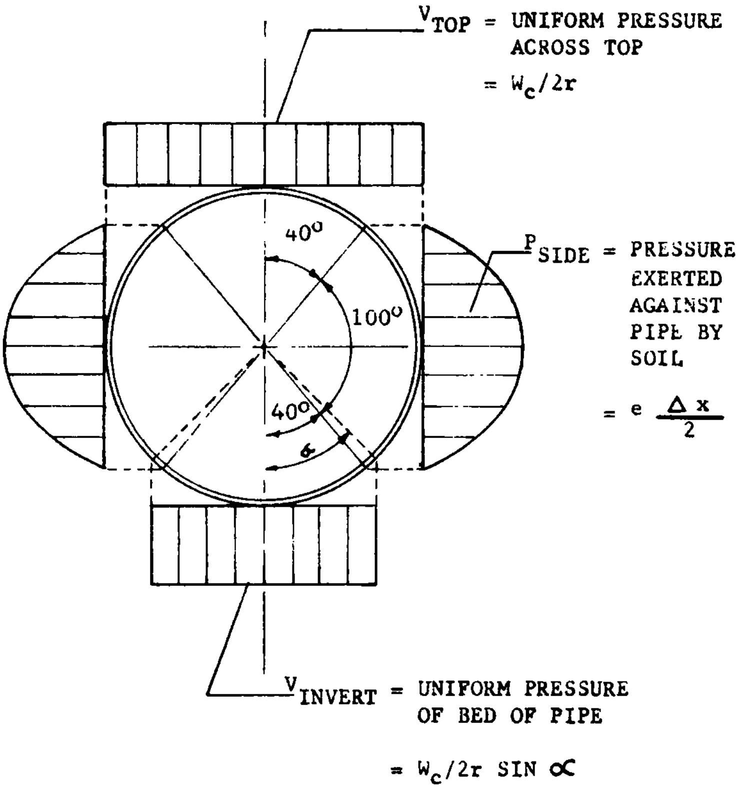


Figure 10. Pressure distribution against pipe per spangler.

Referring once again to Eq. 3, the aluminum culvert equation for test results can be reduced by substitution of known terms: $D_L = 1.00$; $K = 0.108$; $W_c = \frac{1}{12} C_c w B_c^2$ (in which $B_c = 2r/12$ and $w = 135$ pcf x 0.80 = 109, and therefore, $W_c = 0.252\ C_c r^2$); $E = 10 \times 10^6$ psi; and $I = 0.0332t$ in. 4/in. ($2\frac{2}{3}$- by $\frac{1}{2}$-in. pattern) with t = thickness of sheet (in.).

The modulus of passive resistance, E', can be determined from the pressures developed in the soil and the deflection of the pipe into the soil. During the structural deep fill tests both values were recorded on the 48- and 60-in. sizes. These are shown in Fig. 9 from calculation of $e = \frac{P}{\Delta x/2}$.

The results of Eq. 3 are given in Table 4. For computation, C_c is taken from the pressure data previously developed with a typical settlement-projection ratio of -0.50. The comparative analysis deflection compares well with the measured deflection, particularly after all allowances are made for the evident variations in soil compaction and inaccuracies in measurement and prediction of pressures. The evaluation is considered accurate for further use of the basic equation in development of design fill values.

Using the accepted standard soil conditions for design purposes, Eq. 3 can be reformed by substituting Δx = 5 percent of diameter = 0.10r; $D_L = 1.50$, as suggested by Spangler; $K = 0.110$; $W_c = \frac{1}{12} C_c w B_c^2$ (in which $B_c = 2r/12$ and $w = 100$ pcf, and therefore, $W_c = 0.231\ C_c r^2$); $E = 10 \times 10^6$; $I = 0.0332t$; and $E' = 700$ psi to form:

$$C_c = \frac{112}{r} + 0.870 \times 10^6 \frac{t}{r^4} \qquad (4)$$

TABLE 4

CALCULATED AND MEASURED DEFLECTIONS OF ALUMINUM CULVERT PIPE

Culvert Diameter (in.)	Thickness (in.)	Fill (ft)	$\frac{H}{B_c}$	C_c	Δx Calculated	Δx Measured
24	0.060	5	2.5	2.5	0.217	0.12
		10	5.0	4.3	0.374	0.25
		15	7.5	5.6	0.486	0.37
		20	10	6.8	0.590	0.37
		25	12.5	8.2	0.712	0.37
		30	15	9.5	0.825	0.50
24	0.075	5	2.5	2.5	0.206	0.06
		10	5.0	4.3	0.354	0.31
		15	7.5	5.6	0.461	0.44
		20	10	6.8	0.560	0.81
		25	12.5	8.2	0.675	0.94
		30	15	9.5	0.782	1.06
36	0.105	5	1.7	1.5	0.321	0.12
		10	3.3	2.4	0.514	0.18
		15	5.0	3.3	0.707	0.50
		20	6.7	4.4	0.942	0.75
		25	8.3	5.3	1.133	1.00
		30	10	6.3	1.350	1.00
48	0.105	5	1.3	1.1	0.450	0.50
		10	2.5	1.9	0.778	0.68
		15	3.8	2.6	1.064	1.12
		20	5.0	3.3	1.35	1.62
		25	6.2	4.0	1.64	2.62
		30	7.5	4.8		
60	0.135	5	1.0	0.9	0.586	0.25
		10	2.0	1.6	1.043	0.50
		15	3.0	2.2	1.43	0.68
		20	4.0	2.7	1.76	1.00
		25	5.0	3.3	2.14	1.50
		30	6.0	3.9	2.54	2.75

[a]Using value of r_{sp} -0.50.

Once C_c is determined from this equation, a settlement-projection ratio must be assumed from which a fill height-pipe diameter ratio and finally fill height can be evolved. The most severe design requirement would be the complete projection condition of high positive settlement-projection ratio. It is probable, however, that the settlement-projection ratio in practice will not reach this value and only rarely reach +0.5. Normal design tables for aluminum culvert herein are then based on the settlement ratio of zero in conformance with standard practice on other materials. Actually, such a decision builds into the aluminum culvert a safety margin somewhat higher than for the other materials due to the strong tendency toward ditch condition loading. This margin has been considered in later selection and evaluation of safety factors on pipe stress. To show the range possible in evaluation of culvert support strength, the range of fill heights between complete ditch condition and complete projection condition is given in Table 5. The projection values become of interest in the review of vertical elongation at a later point.

Spangler noted that though he was able to confirm the deflection theory with field experience the predicted stresses exceeded the probable stress levels by a wide margin.

TABLE 5

THEORETICAL FILL HEIGHTS AS DEVELOPED FROM SETTLEMENT RATIO THEORY OVER RANGE FROM COMPLETE DITCH TO COMPLETE PROJECTION CONDITION

Culvert Diameter (in.)	Thickness (in.)	C_c	Complete Ditch (r_{sp} = -2.0)		Imperfect Ditch (r_{sp} = -0.5)		Static Weight (r_{sp} = 0)		Imperfect Projection (r_{sp} = +0.5)		Complete Projection	
			$\frac{H}{B_c}$	H	$\frac{H}{B_c}$	H	$\frac{H}{B_c}$	H	$\frac{H}{B_c}$	H	$\frac{H}{B_c}$	H
15	0.060	28.95					28.95	36.2				
	0.075	32.55					32.55	40.7				
18	0.060	19.22			31.8	47.7	19.22	28.8	12.8	19.2	6.9	10.3
	0.075	21.00			35	52.5	21.00	31.4	14	21.0	7.4	11.1
21	0.060	14.30			23.6	41.3	14.30	25.0	9.5	16.6	5.5	9.6
	0.075	15.28			25.3	44.2	15.28	26.8	10.1	17.7	5.7	10.0
24	0.060	11.47			19.0	38.0	11.47	22.9	7.6	15.2	4.65	9.3
	0.075	12.05			20.0	40.0	12.05	24.1	7.9	15.8	4.80	9.6
30	0.075	8.54			13.8	34.5	8.54	21.3	5.5	13.7	3.85	9.6
	0.105	9.03			14.8	37.0	9.03	22.5	5.9	14.7	4.00	10.0
36	0.075	6.73			10.8	32.4	6.73	20.2	4.3	12.9	3.30	9.9
	0.105	6.96			11.3	33.9	6.96	20.9	4.5	13.5	3.4	10.2
	0.135	7.20			11.6	34.8	7.20	21.6	4.7	14.1	3.45	10.3
42	0.105	5.71	17.0	59.4	9.1	31.8	5.71	20.0	3.7	12.9	3.0	10.5
	0.135	5.85	17.5	61.2	9.4	32.8	5.85	20.5	3.8	13.3	3.05	10.7
48	0.105	4.88	13.6	54.4	7.8	31.2	4.88	19.5	3.1	12.4	2.65	10.6
	0.135	4.96	14.2	56.8	8.0	32.0	4.96	19.8	3.2	12.8	2.70	10.8
54	0.105	4.28	13.3	59.9	6.7	30.2	4.28	19.2	2.75	12.4	2.45	11.0
	0.135	4.32	13.4	60.3	6.8	30.6	4.32	19.4	2.8	12.6	2.45	11.0
	0.165	4.37	13.4	60.3	6.8	30.6	4.37	19.7	2.8	12.6	2.50	11.2
60	0.135	3.84	9.4	47.0	6.0	30.0	3.84	19.2	2.45	12.2	2.25	11.2
	0.165	3.87	9.4	47.0	6.0	30.0	3.87	19.3	2.47	12.3	2.25	11.2
66	0.135	3.47	8.1	44.5	5.45	30.0	3.47	19.1	2.2	12.1	2.10	11.5
	0.165	3.49	8.1	44.5	5.45	30.0	3.49	19.2	2.2	12.1	2.1	11.5
72	0.165	3.16	6.4	38.4	4.8	28.8	3.16	19.0	2.0	12.0	2.0	12.0

Strain and pressure data results on the test confirmed this conclusion. However, the experimental data also indicated the probable cause for the disparity in the stress data. If the load distribution on the pipe is modified to reflect a variance in applied pressure across the top and bottom planes, the solution of the deflection equation and determination of moments, reaction, and stresses produced results somewhat more consistent with the experimental results. Assuming a modified pressure distribution across the planes of the installation as shown in Figure 11, the equation for the moment at any point can be written as

$$M_{r\varphi} = M_A + R_A r\,(1 - \cos\varphi) - P_A' \frac{r^2}{2} \sin^2\varphi - P_A'A \frac{r^2}{6} (\sin^3\varphi - 3 \sin^2\varphi) - P_B \frac{r^2}{6} (1 - \cos\varphi)^3 - P_B \frac{r^2}{3} \cos^3\varphi \tag{5}$$

in which

$$\text{Limits} = 0 < \varphi < \pi \text{ and } \pi/2 < \varphi < \varphi < \pi$$

$$A = (P_A' - P_A)/P_A'$$

Solution for moment and deflection produces

$$M_A = P_A' r^2 (0.250 - 0.179A) - 0.155\, P_B r^2 \tag{6a}$$

$$R_A = 0.50\, P_B r^2 \tag{6b}$$

$$\Delta x = \frac{J W_c r^3}{EI + 0.054\, e r^4} \tag{6c}$$

$$J = \frac{0.0835 - 0.055A}{1 - 0.50A} \tag{6d}$$

The results of this analysis confirm the equation developed by Spangler, deviating by a few percent due to the triangular pressure distribution assumption as contrasted to the parabolic distribution of Spangler. Study of the results of these equations indicates the prospective error in results by using triangular and trapezoidal distribution will introduce negligible errors.

In comparing Eq. 6 with the experimental results on 48- and 60-in. pipes, the deflections and pressures (P_A, P_A', and P_B) are given in Table 6. Comparison of results for pressure is based on use of measured total bending and compressive stress as a means to evalutate pressures. The pressures P_A' and P_B were held and P_A varied as required to develop the solution.

The results, though preliminary, show conclusively that by modification of applied pressure distribution to the pipe the predicted and measured stresses can be brought into line and the deflection remains virtually unchanged. This would indicate the approach of Spangler to be valid for description of performance of flexible culvert pipe. Unfortunately, prediction of results is difficult when based on pressures because a small variation in pressure produces a large difference in moment and hence stress.

STRESS EVALUATION

The culvert pipe in the structural test was installed with a large number of strain gauges placed to indicate load strain and hence stress in bending and compression at the quarter points. Before discussion of results can commence, it is necessary to establish the design stress levels that may be used for compression purposes.

The culvert material used in the test was typical culvert sheet having the mechanical properties given in Table 7.

$$W_C = (P_{A'} + P_A)\, r$$

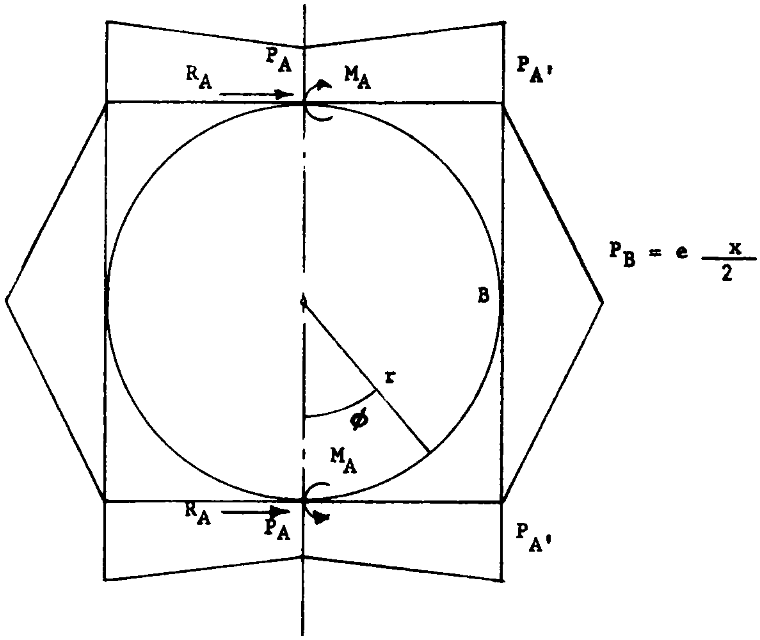


Figure 11. Modified pressure distribution across planes of installation.

The selection of a suitable safety factor shall be made by application of the design values proposed for an alloy of similar mechanical properties in the ASCE Specification for Structures of Aluminum Alloy 6061-T6 (4). This specification covering an alloy with ultimate strength of 38,000 psi and yield strength of 35,000 psi is as follows: safety factor on ultimate strength = 2.71; safety factor on yield strength in tension = 2.33; and safety factor on yield strength in compression and buckling = 2.50. These values are further explained in Part IV, Section A, Summary of Allowable Stresses, Item A-1:

TABLE 6

MEASURED AND THEORETICAL PRESSURES ON 48- AND 60-IN. SECTIONS

Pressure Type	48-In. Culvert, 24-Ft Fill (psi)	60-In. Culvert, 30-Ft Fill (psi)
Meas. side pressure P_B	21	17
Meas. pressure directly over pipe P_A	5.2	6
Pressure on adjacent soil P_A at pipe top	18.5	22.8
Meas. avg. max. stress in pipe	20,000	25,000
P_A computed pressure over pipe[a]	12.9	7.8

[a]Based on measured P_B, P'_A, stress.

TABLE 7

MECHANICAL PROPERTIES OF TYPICAL CULVERT SHEET

Type	Ultimate Tensile Strength (psi)	Yield Strength (psi)	Elongation in 2 In. (%)
Flat corrugated sheet	35,000	29,000	9
Culvert product	37,000	32,000	
Recom. design min. properties of completed culvert	34,000	28,000	

<u>A-1. Basic Tensile Design Stress.</u>-- The basic tensile design stress of 15 kips per sq in. represents a factor of safety of 2.33 based on the specified tensile yield strength. This is a larger factor of safety with respect to yield strength than is ordinarily encountered in specifications for structural steel. In selecting this rather large factor of safety on yield strength, the committee was influenced to a considerable extent by the fact that there is a smaller spread between yield strength and tensile strength in this aluminum alloy than is commonly encountered in structural steels.

Using the preceding values, a design maximum stress for combined bending and axial compression is S = 11,200 psi, which is used to develop the fill heights from the strain data on the pipe.

A proper value of the safety factor for the resistance to rivet crushing, shear, or pull-out can be developed from the ultimate joint strength. This approach is similar to the well-known compression ring theory. The computed strength of the rivet joint is based on a shear strength of 18,500 psi and an ultimate bearing strength on the sheet of 65,000 psi. It is suggested that for design purposes a safety factor of 3.3 be used to set the crushing design limits or a value lower if that value is consistent with basic steel pipe design practices. This allows for a normal safety factor of 3.0 with a 10 percent reduction to account for the increase of typical properties over minimum properties of the joint. Similarly, if spot welding is to be used a safety factor of 3.3 is suggested to apply against typical spot strengths. This value is consistent with design considerations for other components of the system, and considering that where deep fill design is encountered, excellent control of construction with attendant accuracy of prediction of load, the safety factor is considered ample.

Evaluation of safety factors in design based on soil factors is difficult. It is important to distinguish between the development of load on the pipe due to the soil settlement and internal shear and performance of the soil-pipe combination once this load has been established. Safety factors are applied to performance under load and judgement governs the load application. It is accepted practice to consider a final deflection of 5 percent of the diameter to the design limit. This is based on the assumption that 20 percent deflection is tantamount to failure with an apparent safety factor of 4.0 to arrive at 5 percent.

RESULTS

There are several methods of evaluation of flexible metal pipe with respect to its load-carrying ability. Three theoretical approaches may be used to evaluate the predicted load-carrying ability: elastic instability, crushing, and deflection analysis.

These may now be corroborated by comparing measured deflection and stress data obtained and from the combined strain evaluation, an accurate basis for fill height table determination for corrugated aluminum culvert may be prepared.

Full Circle Pipe

The basic method of evaluation of pipe is in the full circle form. This lends itself to analysis of the structure using the assumption that there is little or no initial stress in the pipe. Load-carrying ability of flexible pipe, and aluminum pipe in particular, is critically dependent on the distribution of load and the total load or fill height. It is necessary then to make a basic assumption of the magnitude of load before completing the analysis. The worst design load condition occurs when complete projection is assumed. To attain this condition physically, the soil must be so loosely filled about the pipe that the soil adjacent to the pipe can compress substantially more than the culvert. Practically, this condition is difficult to attain in flexible pipe, and if attained at all, service of the pipe would be hazardous because of the very low compaction and the pipe would compress out of round readily. The probable ultimate limit design condition to be encountered in the field would be the case where the soil would compress twice as much as the pipe resulting in a settlement-projection ratio of +0.5, but even this is unlikely. For design purposes it is best to assume the condition where the entire weight of soil and live load directly above the pipe is carried directly to the pipe. This is the case where the settlement-projection ratio is zero or $C_c = H/B_c$, resulting in the load on the pipe being computed from $W = wHB_c$. This condition is used for the design fill analysis comparison on full circle pipe.

Having established the design load conditions, the test results and prediction equations may now be compared. Table 8 gives the composite result from which a fill table for full circle pipe can be extended.

A series of 14 stations were used to evaluate performance of culvert pipe. Strain curves were developed into bending and compressive stress curves as load progressed. Using design stresses and safety factors the recommended maximum fills for bending, joint compression, and deflection can be evaluated.

In summing up the results of the several methods of evaluation of full circle pipe and comparing this with experimental data, a normal design fill height can be developed for each diameter and gauge. In the selection of the height it is necessary to consider the relative weight or validity of each set of predictions and test results. The crushing or shear concept serves as an excellent check which the design levels should not exceed, but it is not the major loading condition experienced in full circle pipe as reflected from this test. The prime weight should be given to the deflection or moment computation and the stress values determined. The final selection of the fill height is shown in the last column of Table 8 and serves the basis for prediction of the fill height for all diameters and gauges. Extrapolation to sizes other than tested can be made by extension of the deflection equation.

Taking a specific look at the deep fill results shows some disparity from a precisely normal pattern. For example, the 36-in. pipe consistently performed better than average, whereas the 24-in. one was equally below average. The 48- and the 60-in. ones were about as expected. These differences can readily be attributed to differences in compaction around the pipe and settlement of the soil adjacent to the pipe as a small difference in either can produce a substantial difference in results.

However, the placing of the pipe followed acceptable good practice with nominal effort being made to produce better-than-average conditions. The conditions of the test are typical of average requirements of compactable soil and reasonable backfill compaction around the pipe. The results shown will hold for compaction around the pipe of 80 percent (by the Proctor method) or more. When the compaction drops below 75 percent the design fill height or load-carrying ability is severely restricted. Simply stated, no flexible or rigid culvert can be expected to carry significant load without some compaction of backfill material.

Just as reduction in compaction will cause a reduction in load capacity, closely controlled good compaction will result in greater ability to carry load. When the backfill

TABLE 8

DESIGN FILL HEIGHTS BY ANALYTICAL AND MEASURED MEANS[a]

Culvert Diameter (in.)	Thickness (in.)	Fill Height: Elastic Instability-Determined[b]	Fill Height: Crushing Or Shear-Determined[c]	Fill Height: Deflection Settlement-Determined $r_s p = 0$	Fill Height: Deflection Settlement-Determined $r_s p + = 0.5$	Mean Compressive Stress for Shear from Strain Gauges (fig.)	Extreme Fiber Stress From Strain Gauges (fig.)	Recommended Design Fill Height for Aluminum Alloy Full Circle Culvert
24	0.060	8	15	22	15	12	12	13
						16	19	
						--	18	
	0.075	16	24	24	16	20	19	21
36	0.105	10	18	21	13	22	15	18
						21	27	
						28	19	
48	0.105	6	24	19	12	--	15	15
						26	15	
						36	18	
60	0.135	4	20	19	12	18	12	14
						22	15	

[a] Rivet sizes are 5/16-in. diameter for 0.060- and 0.075-in. sheet and 3/8-in. diameter for 0.105- and 0.135-in. sheet.
[b] See (1) for derivation.
[c] See (1) for derivation. Safety factor of 3.3 or published value used.

TABLE 9

FILL HEIGHT COMPARISONS AND RECOMMENDATIONS VERTICALLY ELONGATED CULVERT

Culvert Diameter (in.)	Thickness (in.)	Crushing or Shear Fill Height Based on Load at		Deflection Settlement–Determined Fill Height		Recommended Fill Height	
		$r_s p = 0$	$r_s p = -0.50$	$r_s p = -0.20$	$r_s p = -0.50$	Vertically Elongated Only	Prestress Strutting
36	0.105	17	27	26	33	18	27
	0.135	18	29	27	34	18	29
42	0.105	29	47	24	31	20	31
	0.135	29	47	25	32	25	32
48	0.105	25	41	24	31	18	31
	0.135	25	41	26	33	21	33
	0.165	25	41	23	30	23	30
54	0.105	22	36	23	30	18	25
	0.135	22	36	23	30	20	30
	0.165	22	36	23	30	22	30
60	0.135	20	32	23	30	17	25
	0.165	20	32	23	30	20	25
66	0.135	18	29	23	30	15	25
	0.165	18	29	23	30	18	25
72	0.135	17	28	23	30	13	20
	0.165	17	28	23	30	17	20
78	0.165	16	26	23	30	12	16
84	0.165	15	24	23	30	10	15

is placed in accordance with the best specifications and inspected at 90 to 95 percent Proctor density, the load-carrying capacity of aluminum pipe can be expected to increase. The design mechanism to predict this increase would be to select a high negative settlement-projection ratio or to use the ditch condition analysis. A good approximation of this would be to decrease the settlement ratio from 0 to -0.40, the conditions experienced in these tests with only 80 percent compaction. This approach would increase load capacity of aluminum pipe by approximately a factor of about 1.25. It should be emphasized, however, that design in this range must be based on thorough knowledge of the soil properties and degree of compaction and continuous inspection of the installation.

Vertically Elongated Pipe

Vertically elongated pipe (Fig. 12a) is defined as that pipe which is distorted from the full circle shape by elongating the vertical axis and, consequently, shortening the horizontal axis an equal amount. There are three accepted methods of elongation, each with a clearly defined purpose to improve the load-carrying ability. The three means shown in Figure 12b, 12c, and 12d are factory elongation without struts or stays, factory elongation with wire struts to preload the pipe, and field elongation vertical with struts to preload the pipe. Factory elongation is usually accomplished by compressing the pipe and twisting wire strutting at nominal 24-in. spacing. Field vertical strutting is usually done with timber cap and sole runners and column spacing at 5- to 6-ft centers placed by jacking or wedging. Each method of elongation produces an improvement in load-carrying ability which can be approximated.

When pipe is elongated by permanent deformation up to 5 percent, the general stress behavior under load is approximately the same as if it were full circle. There is some initial prestress in the pipe metal of a sign opposite that of the loading due to plastic flow in forming so that the actual exteme fiber stress at any load condition is somewhat less than the values that would be indicated. However, in normal cases this prestress is low, typically less than 3,000 psi, and is not included in calculations. Having established that the load-carrying analysis of factory-elongated pipe is nearly the same as for full circle pipe, additional fill height capabilities must be attainable by means other than simple stress or deflection. This is done in two related ways. First, the pipe can settle up to 5 percent and still be in the round shape. This allows 5 percent of the diameter to be extended into the soil at the side to improve the soil modulus of passive resistance markedly without signs of distortion. In properly compacted soils this would also serve to make the soil adjacent to the pipe much less liable to settlement, thus producing a high negative settlement-projection ratio and distributing the fill load to the surrounding soil. In ordinary compacted soils, the initial expansion can occur with low stress due to the lower passive modulus allowing the pipe to take the new position without high stress and then be in position to absorb stress from higher loads beyond this point. In summary, vertically elongated culvert should have load- or fill-carrying capacity estimated to be 25 percent more than full circle in normal (80 percent compaction) soils. Comparing this with the basic fill height table with a settlement ratio of zero, the increase would be equivalent to the fill capacity with a settlement ratio of -0.20 and essentially the same as full circle pipe with 95 percent compaction. Vertical elongation would be expected to contribute little where soil compaction of 95 percent is attained except that the initial shape offers opportunity to provide continuity of high load-carrying ability if a few relatively soft pockets might be encountered. One cannot expect to attain higher than 25 percent improvement by elongation and control of soil as the stress developed in the pipe will effectively limit further gain.

When vertical elongation is accomplished by field timber strutting or by shop or field wire strutting by tensioning, the analysis of behavior changes somewhat to further improve the load-carrying ability of the pipe. In addition to the improvement noted, due to more negative settlement-projection ratio the pipe is no longer limited by developed bending stresses. In the elongation process, bending moments are impressed into the structure in accordance with the equations shown on Figure 12. Max-

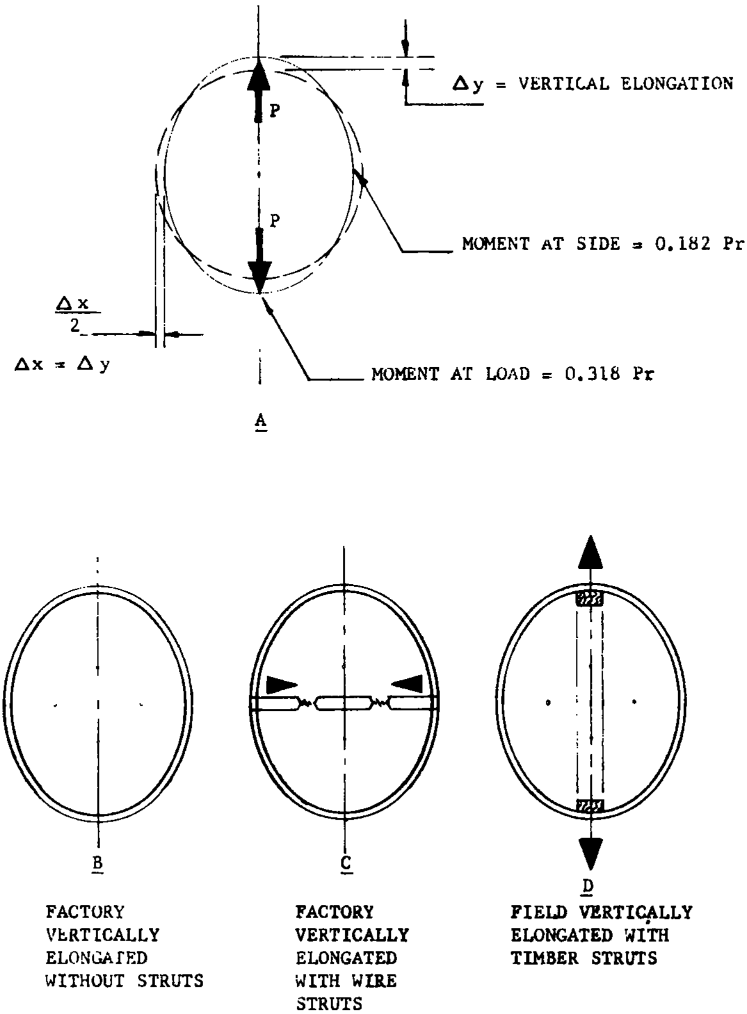


Figure 12. Vertically elongated pipe.

imum stress developed in aluminum alloy culvert is limited to about 26,000 psi, the elastic limit.

Taking a typical analysis of 60-in. pipe of 0.135-in. thickness with a maximum strutting prestress of 22,400 psi at the point of load and 12,900 psi at the side, the stress condition under load can be approximated by superimposing the prestresses on the load stresses developed in the test. Referring to the measured stress, the maximum pipe stress under 14 ft of load will be approximately 11,200 psi in bending. If it is assumed that timber strutting has been used, the extreme bending stress at the side is now +12,900 - 11,200 psi or +700 psi. If the loading stress is extended by linear means to a point where the combined prestress and load stress reach the maximum design bending stress, the fill height determined by bending stress would be approximately doubled.

If the analysis by strutting is transferred to its effect on the soil around the structure, marked improvement in performance can again be predicted. The struts are usually left in place until the fill is completed, allowing the load pressure to compact the soil around the pipe further. When the struts are finally removed, the expansion of the pipe into this compacted area coupled with equal settlement immediately over the pipe will cause a high negative settlement ratio to occur, perhaps approaching complete ditching condition. In addition, a higher modulus of passive resistance can be expected due to the high percent of compaction and lack of compressibility. In general, the decrease in settlement ratio and increase in modulus of passive resistance could be expected to produce design fill heights of nearly double the full circle values reported for prestress-strutted pipe.

Considering that the effect of bending stress under high load now becomes small for prestress-elongated pipe the shear or crushing stress analysis becomes the governing design condition. Maximum load-carrying ability of prestress-elongated culvert becomes simply an analysis relating soil load to joint strength by means of a suitable safety factor analysis (Table 9). In the installation of elongated pipe it is important that careful control of compaction is exercised so that maximum advantage be taken of the settlement ratio or the reasons for use of elongated pipe are wasted. With all this care in the installation and with provisions for deflection lag it is felt that the earlier safety factor of 3.3 can still be applied to ultimate joint strength through shear or crushing.

Concentration Loads

As a part of the complete program on evaluating aluminum culvert, a series of concentration load tests were run on sections of 24-in. diameter 0.060-in. sheet and 36-in. diameter 0.075-in. sheet sections. Fills of 1, 2, and 3.5 ft were placed and compacted over the pipe with an average compaction of 87 percent Proctor density. Load distribution causing pressure in the soil and stress in the pipe was developed through a series of steel plates, 15, 18, and 24 in. square, used to support one side of a D7 loader with a reaction of 19,350 lb directly over the pipe. Pressures were measured at the top, sides, and invert of the pipe and strain readings were taken in the same six locations in the circle used in the deep fill tests.

The pressures recorded were compared with the predicted pressures that could be encountered at the various points in the soil. As a basis for comparison, pressure distribution by both the method of 45° divergent uniform pressure and the Boussinesq pressure bulb were prepared. These predictions compared well with the measured pressures. After review of the pressure distribution it was felt that the 45° divergent uniform pressures were sufficiently accurate to use as the standard method of analysis for concentration loading.

The uniform pressure equation is

$$F = P\,(a + 2h)\,(b + 2h) \tag{7}$$

in which

- F = force on plate or surface;
- P = pressure at any point below surface within pressure rectangle;
- a, b = side dimensions of plate; and
- h = depth to pressure.

To evaluate the relationship between surface load, pressure, and strain, each plate size-fill height combination could be converted to an equivalent pressure at the top of the pipe through Eq. 7 and the strain data plotted as a function of pipe top pressure. A typical curve is Figure 13. From these data a smooth curve of strain at the top of the pipe vs pressure at the top of the pipe can be plotted. Using these strain data, stress curves for average compression, crushing or shear, and bending can be shown in Figure 14. Using the safety factor analysis and design stresses of 11,200 psi in bending and 2,100 psi in crushing, critical pipe top pressures of 26.4 and 27.2 psi for the 24-in. and 36-in. ones were developed. Once critical pressure is established (in this case, the standard was selected as 25 psi for the 36-in. diameter), the surface

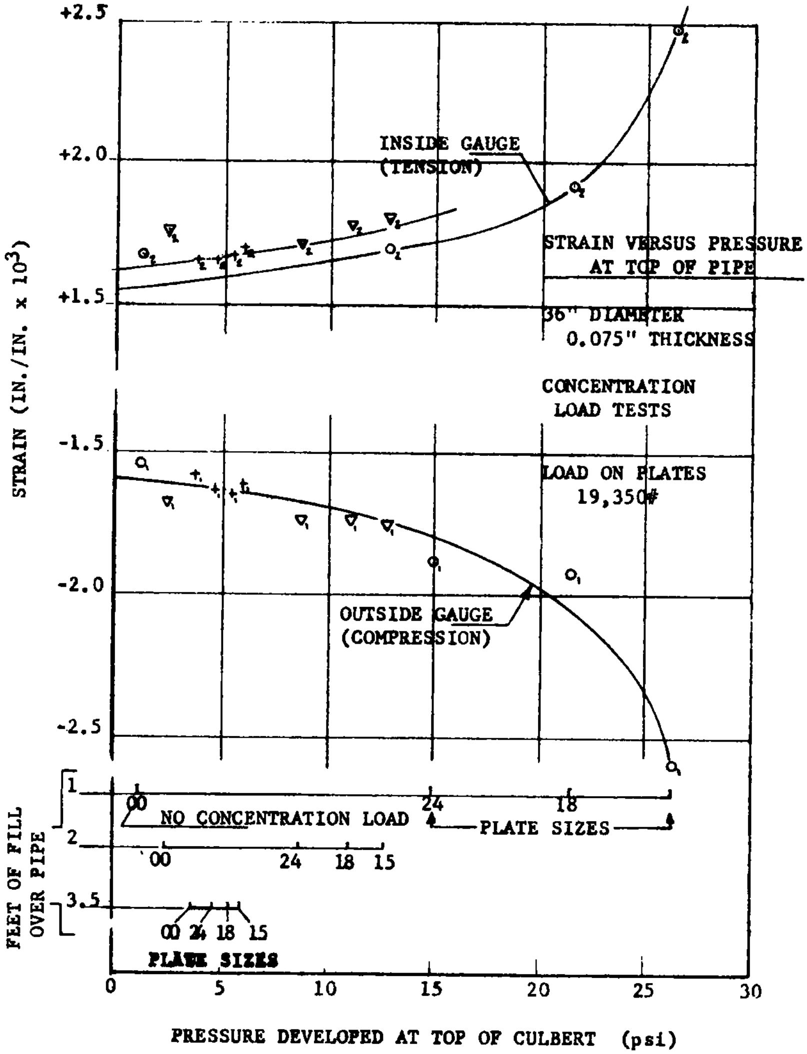


Figure 13. Strain vs pressure at top of pipe.

load for each fill height can be computed to satisfy this condition. Referring to Eq. 7, the following modification produces, assuming a wheel contact area of 6 by 20 in.:

$$F'I = 100\ K(3 + h)(10 + h) \tag{8}$$

in which

- F'= surface load (wheel) in pounds;
- I = impact factor;
- h = depth to top of pipe; and
- K = a factor allowing for change in pipe diameter (1.0 for 36-in. diameter).

Referring to the AASHO Standard Specification for Bridges (5), an impact factor of 1.30 is recommended for culvert with low fill. This value is used in development of a curve relating maximum traffic wheel load to minimum fill height over pipe for compacted and well-graded or paved fill shown as Figure 15. An impact factor of 2.0 was

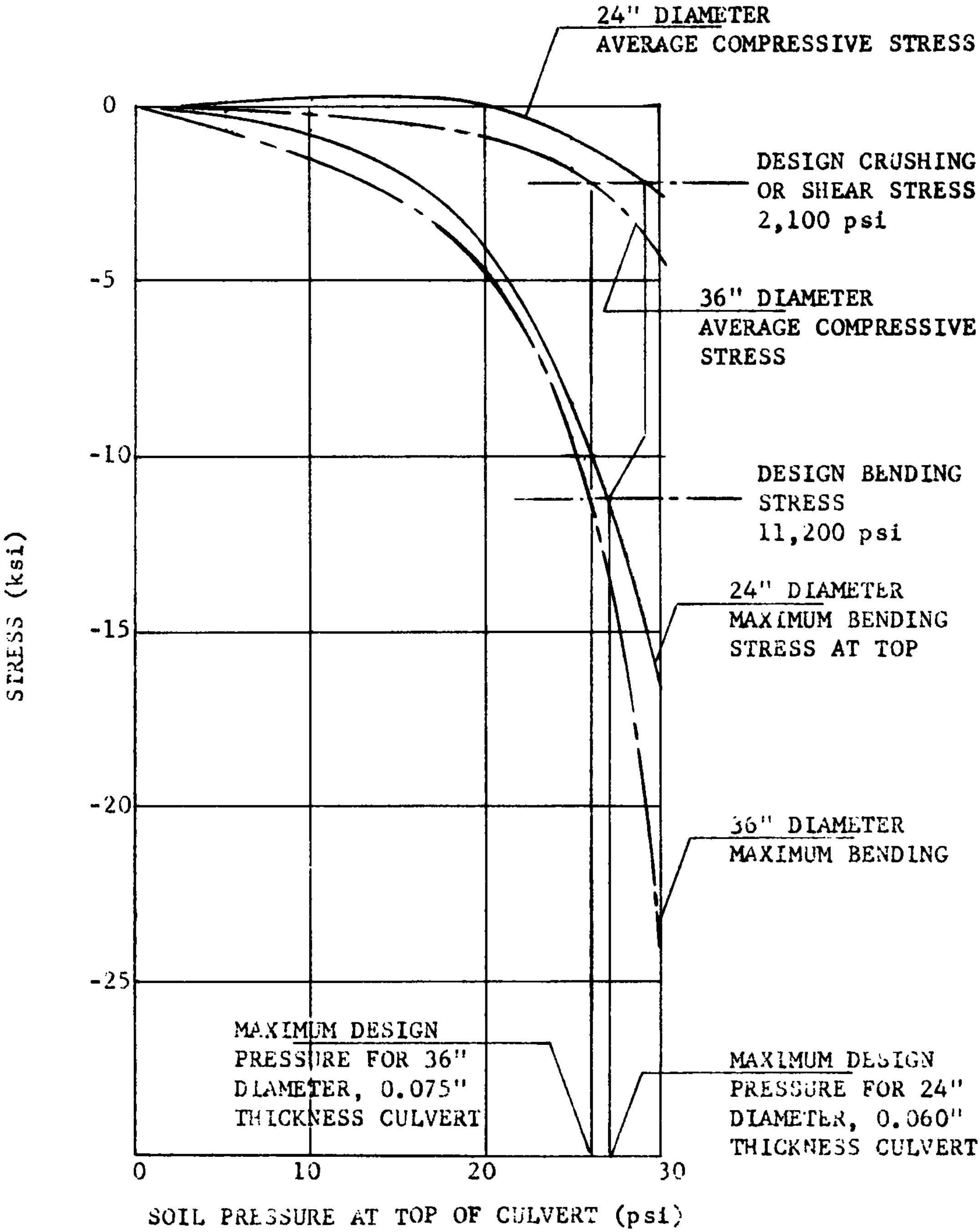


Figure 14. Average compressive stress and maximum bending stress at top of culvert vs pressure at top of culvert.

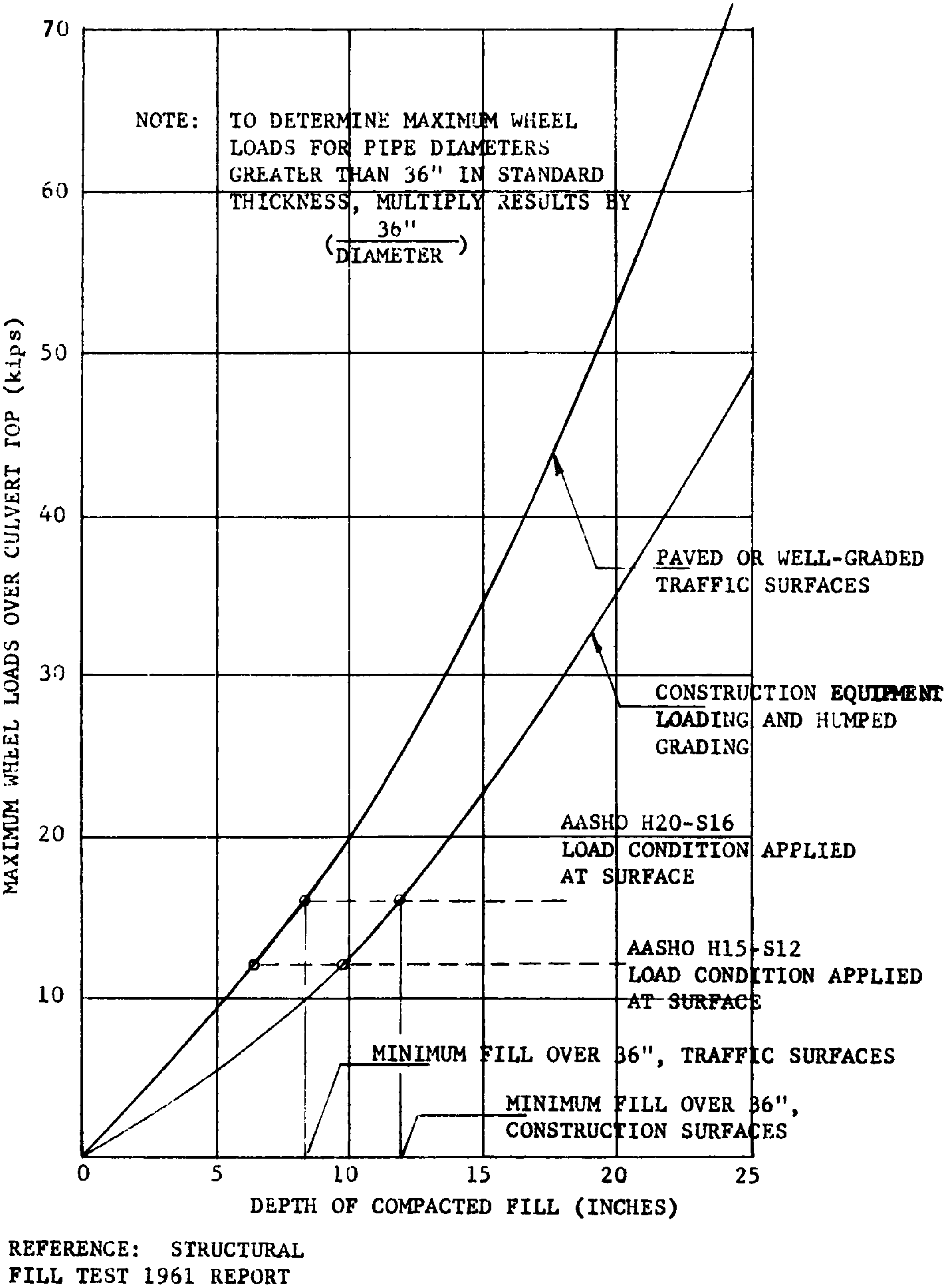


Figure 15. Maximum wheel loads vs fill height over culvert for 36-in. diameter and less.

used in preparation of the second curve which estimates maximum wheel loads over minimum compacted fill during construction or on badly graded surfaces. Although the results shown are approximations, they nevertheless serve as a useful guide to the limiting of loading during construction, a persistent problem in the field. This design level is at best an approximation. Unfortunately, high speeds with large construction equipment may have much higher impact factors and because of the shock nature of loading the distributed area may be substantially smaller. It would be wise to have a minimum cover equal to the pipe diameter.

The minimum fill height must increase as the pipe diameter increases because the loading is in the moment form. The factor K would be expected to be related by the diameter ratio, using 36 in. as standard. However, in the case of 24 in., the multiplier is 1.08 from the tests (see Table 10).

TABLE 10

RELATION OF DIAMETER TO FACTOR K

Diameter (in.)	K
24 and less	1.08
30	1.00
36	1.00
42	0.86
48	0.75
54	0.67
60	0.60
66	0.54
72	0.50

Referring to the curves of wheel load vs fill height, the limit conditions for the well-established AASHO H20-S16 and H15-S12 are shown for conventional highway use to be 8 in. and 6.5 in., respectively, for pipe of 36-in. diameter. The minimum fill for any other size would be determined by multiplying the 8-in. and 6.5-in. values by the reciprocal of K. This serves as the basis for minimum fill recommendations in Table 1.

Neither the stresses nor pressures measured at the sides and invert of the pipe were considered in the previous determination. This is because measured values of both were but a small section of the top load conditions showing conclusively that the load at the top is distributed to the soil rapidly in a beam-type action and that structural considerations need not carry below the top area.

FUTURE WORK

It can generally be concluded from these tests that aluminum alloy culvert pipe is structurally capable of supporting high soil fills and is also capable of resisting wheel or impact loads under low fills.

Future work in the area of flexible culvert should be directed toward a more thorough and detailed knowledge of the behavior of soil pressure and culvert pipe resistance arour the complete circle. These tests would be expected to show a means of establishing better corroboration between deflection and stress evaluation. Concurrently, pressure traverses across the pipe top are necessary to establish exact patterns of pressure distribution. Finally, strain readings taken during the entire cycle of strutting and soil loading, first, with struts in place to completion of fill and, second, with struts removed under partial fill, should be evaluated.

REFERENCES

1. "Technical Information Report on Aluminum Culvert." Kaiser Aluminum and Chemical Sales, Inc., (Jan. 1961).
2. Spangler, M.G., "The Structural Design of Flexible Pipe Culverts." Iowa State College Bull. 153 (1941).
3. Spangler, M.G., "Soil Engineering." 2nd ed., International Textbook Co. (1960).
4. "ASCE Specification for Structures of Aluminum Alloy 6061-T6." Paper 970, Jour. Struc. Div., ASCE, 82:ST3 (May 1956).
5. "Standard Specification for Highway Bridges." 7th ed., AASHO (1957).

6. "Recommended Standards for Wrought Aluminum Products." Aluminum Assoc.
7. Timoshenko, S., "Theory of Elastic Instability." McGraw-Hill (1936).
8. White, H. L., and Layer, J. P., "The Corrugated Metal Conduit as a Compression Ring." HRB Proc., 39:389-397 (1960).

Discussion

SOME DO'S AND DONT'S IN THE DESIGN AND INSTALLATION OF FLEXIBLE PIPE CULVERTS

M. G. SPANGLER, Research Professor, Iowa State University, Ames—These remarks deal solely with the mechanics of the supporting strength of flexible pipe culverts and have no reference to the kind of material of which such conduits are made. They are equally applicable to pipes of any material, whether corrugated iron or steel, or welded smooth steel pipe, just so they are flexible in character.

Approximately 25 years ago the writer made rather extensive measurements of the magnitude and distribution of loads and pressures on flexible pipes under earth cover. The results of these measurements led to the development of the following load hypothesis for this type of structure:

1. The vertical load on a pipe may be determined by Marston's theory of loads on conduits and is distributed approximately uniformly over the breadth of the pipe.
2. The vertical reaction on the bottom of the pipe is equal to the vertical load and is distributed approximately uniformly over the width of bedding of the pipe.
3. The passive horizontal pressures on the sides of a pipe are distributed parabolically over the middle 100^0 of the pipe, and the maximum unit pressure is equal to the modulus of passive resistance pressure of the sidefill material multiplied by one-half the horizontal deflection of the pipe.

This hypothesis is shown graphically in Figure 16. It is emphasized that the pressures on the sides of the pipe, which hold the structure in equilibrium, are passive pressures. They do not and cannot develop unless and until the pipe deflects and moves outward against the sidefills an amount sufficient to mobilize the passive resistance characteristics of the enveloping soil. In other words, the pipe must deflect and the sides must move outward against the soil in order to develop the lateral pressures required for equilibrium.

Measurements of radial pressures on two of the experimental pipes together with pipe deflections, are shown in Figures 17 and 18. It is indicated that these pressures are essentially uniformly distributed around the periphery of the pipe. They appear to be quasi-hydrostatic in character. Other experimenters have obtained similar results. This uniform pressure situation has led some observers to conclude that the stresses in the pipe wall are compressive stresses only, as they would be in a circular vessel subjected to hydrostatic pressure. This conclusion is erroneous, in the writer's opinion. It must be realized that the pipes had to deflect for this quasi-hydrostatic pressure to develop, and when a

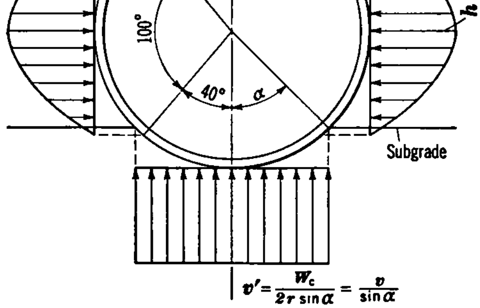


Figure 16. Hypothetical load system on flexible pipe culvert.

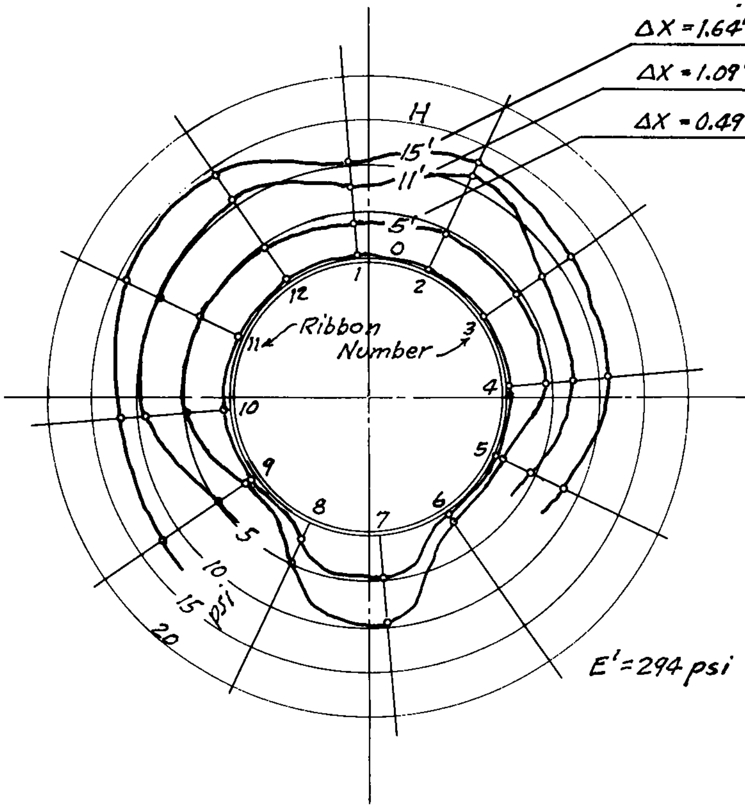


Figure 17. Radial pressures and deflections, 42 in. 8 ga. pipe, experiment 1.

pipe deflects there is bending moment in the pipe wall. Therefore, a flexible pipe ring under earth load is subjected to a combination of thrust and bending moment, not just thrust alone. There are radial shears in the pipe wall also, but these are relatively unimportant.

Under the load and stress situation postulated, flexible pipes tend to progress toward failure by one or both of two types of structural action. One of these is deflection of the pipe ring. As the vertical earth load and the equal and opposite vertical reaction on the bottom of the pipe increase, the pipe deflects; that is, the vertical diameter shortens and the horizontal diameter lengthens. The amount of deflection depends on the magnitude of the load, the stiffness of the pipe wall, and the passive resistance characteristics of the enveloping soil. If the deflection is excessive, the pipe will collapse. Therefore a prediction of the amount of deflection that will develop in a given installation is necessary in the design process, and the deflection must be held within tolerable limits.

The other structural action that may lead to failure is the seam stress; that is, the combination of tangential thrust and bending moment stress at bolted or riveted longitudinal lap seams in the pipe wall. In smooth steel casing pipes, such as those used to protect gas and petroleum products transmission lines at railway and highway crossings, these stresses are of no significance because the longitudinal seam is usually a welded joint that is as strong as the pipe wall at any point. However, in the culvert pipes fabricated of corrugated metal sheets, particularly those of the larger diameters, the design of the longitudinal seams to resist the combined tangential thrust and bending moment stress is of very great importance.

Equations have been developed to facilitate investigation of both these types of structural action. In this development, the loaded pipe and the enveloping soil have been

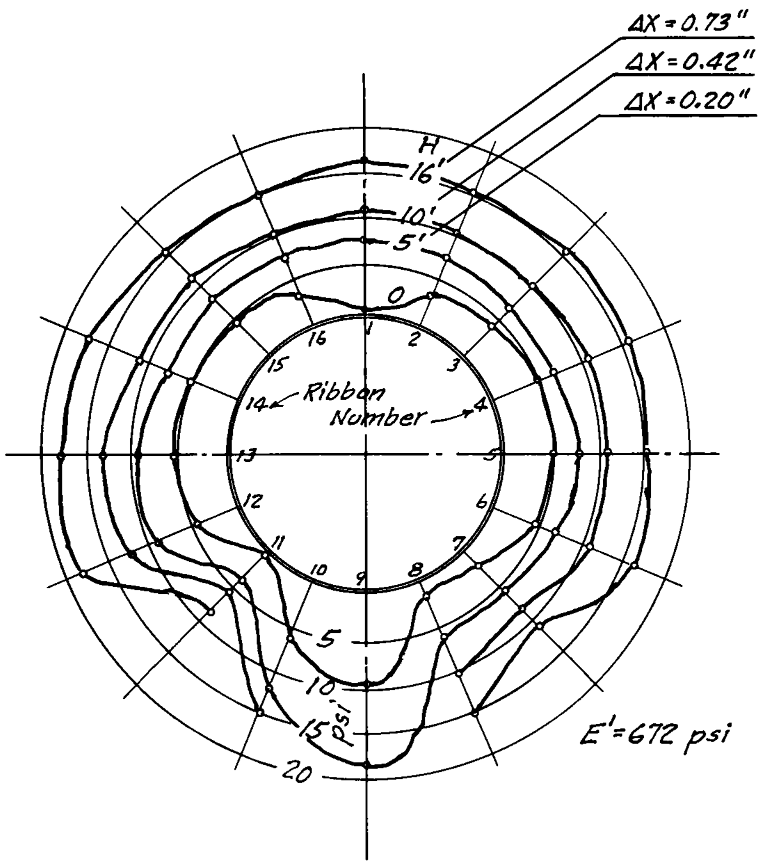


Figure 18. Radial pressures and deflections, 42 in. 10 ga. pipe, experiment 2.

treated as an indeterminate elastic body. The prediction equation for deflection is

$$\Delta x = \frac{D_L K W_c r^3}{EI + 0.061\, E' r^3} \tag{8}$$

in which

Δx = horizontal deflection, in.;
D_L = a deflection lag factor;
K = bedding constant,
W_c = load on pipe, lb per in. of length;
r = mean radius of pipe, in.;
E = modulus of elasticity of pipe metal, psi;
I = moment of inertia of section thru pipe wall, in. 4/in.;
E' = er, modulus of soil reaction, psi; and
e = modulus of passive pressure of soil, psi/in.

Equations for moment and thrust at any point in the pipe wall are moment, thrust and shear due to vertical load (One-half bedding angle, $\alpha = 45°$):

$$0 < \varphi < 45^0 \quad \left| \begin{array}{ll} M_D = W_C r\,(0.183 - 0.026 \cos\varphi \,.\, 0.354 \sin^2\varphi) & (9) \\ R_D = W_C\,(0.026 \cos\varphi + 0.707 \sin^2\varphi) & (10) \\ S_D = W_C\,(0.707 \sin\varphi \cos\varphi - 0.026 \sin\varphi) & (11) \end{array} \right.$$

$$45^\circ < \varphi < 90^\circ \quad \left| \begin{array}{ll} M_D = W_c r\,(0.360 - 0.026 \cos\varphi - 0.500 \sin\varphi) & (12) \\ R_D = W_c\,(0.026 \cos\varphi + 0.500 \sin\varphi) & (13) \\ S_D = W_c\,(0.5 \cos\varphi - 0.026 \sin\varphi) & (14) \end{array} \right.$$

$$90^\circ < \varphi < 180^\circ \quad \left| \begin{array}{ll} M_D = W_c r\,(0.11 - 0.026 \cos\varphi - 0.25 \sin^2\varphi) & (15) \\ R_D = W_c\,(0.026 \cos\varphi + 0.5 \sin^2\varphi) & (16) \\ S_D = W_c\,(0.5 \sin\varphi \cos\varphi - 0.026 \sin\varphi) & (17) \end{array} \right.$$

Moment, thrust and shear due to horizontal load:

$$0 < \varphi < 40^\circ \quad \left| \begin{array}{ll} M_D = hr^2\,(0.345 - 0.511 \cos\varphi) & (18) \\ R_D = 0.511\ hr \cos\varphi & (19) \\ S_D = 0.511\ hr \sin\varphi & (20) \end{array} \right.$$

$$40^\circ < \varphi < 140^\circ \quad \left| \begin{array}{ll} M_D = hr^2\,(0.199 - 0.5 \cos^2\varphi + 0.143 \cos^4\varphi) & (21) \\ R_D = hr\,(\cos^2\varphi - 0.568 \cos^4\varphi) & (22) \\ S_D = hr\,(\sin\varphi \cos\varphi - 0.568 \sin\varphi \cos^3\varphi) & (23) \end{array} \right.$$

$$140^\circ < \varphi < 180^\circ \quad \left| \begin{array}{ll} M_D = hr^2\,(0.345 + 0.511 \cos\varphi) & (24) \\ R_D = -0.511\ hr \cos\varphi & (25) \\ S_D = 0.511\ hr \sin\varphi & (26) \end{array} \right.$$

in which (see Fig. 19)

M_D = bending moment at any point D;
R_D = tangential thrust at any point D;
S_D = radial shear at any point D; and
$h = \dfrac{E' \Delta X}{2r}$

To obtain the moment or thrust at any point on the pipe ring, the values at the point due to vertical load and horizontal pressure should be added algebraically. Moment and thrust diagrams based on these equations are shown in Figure 19. The calculated flexural stress at all points around the periphery of the 42-in. 8 ga. pipe with standard corrugations under 15 ft of fill in experiment 1 are shown in Figure 20. Calculated compressive stresses due to tangential thrust in the pipe wall are also shown in this figure.

The calculated flexural stresses are very much greater than the compressive stresses due to thrust. Also, the flexural stresses are very high and well above the probable elastic limit of the metal. This, of course, does not mean that actual stresses were as high as the calculated values, because the flexure formula is not valid when stresses exceed the elastic limit. It does mean, however, that the metal in the pipe ring was stressed in the plastic range over a substantial portion of the periphery. This conclusion is qualitatively verified by the fact that when the fill soil in this experiment was removed the pipe sections were permanently deformed, indicating that they had been stressed in the plastic range under the fill load.

In spite of the indication that stresses in a flexible pipe culvert may readily exceed the elastic limit of the metal, the writer does not advocate that such pipes should be designed on the basis of stress level. When the pipe metal is stressed in the plastic range it simply means that the rate of deflection of the pipe under load increases, and the sides of the pipe will push harder against the soil. This mobilizes the passive resistance pressure of the soil at a more rapid rate, and equilibrium forces are built up against the sides of the pipe. Also, observations indicate that flexible metal pipes fail by excessive deflection rather than by rupture of the pipe wall. Therefore, a limiting deflection of the pipe rather than a limiting stress level is believed to be the most import criterion for design.

The corrugated metal pipe industry has long advocated a deflection limit of 5 percent of the nominal pipe diameter as a design criterion and the writer's experience tends to indicate that this figure is satisfactory. However, in the American Society of Civil Engineers Manual of Practice 37 (Water Pollution Control Federation Manual of Practice No. 9) on "Design and Construction of Sanitary and Storm Sewers," a factor of safety of 1.25 is applied to the limiting deflection of 5 percent.

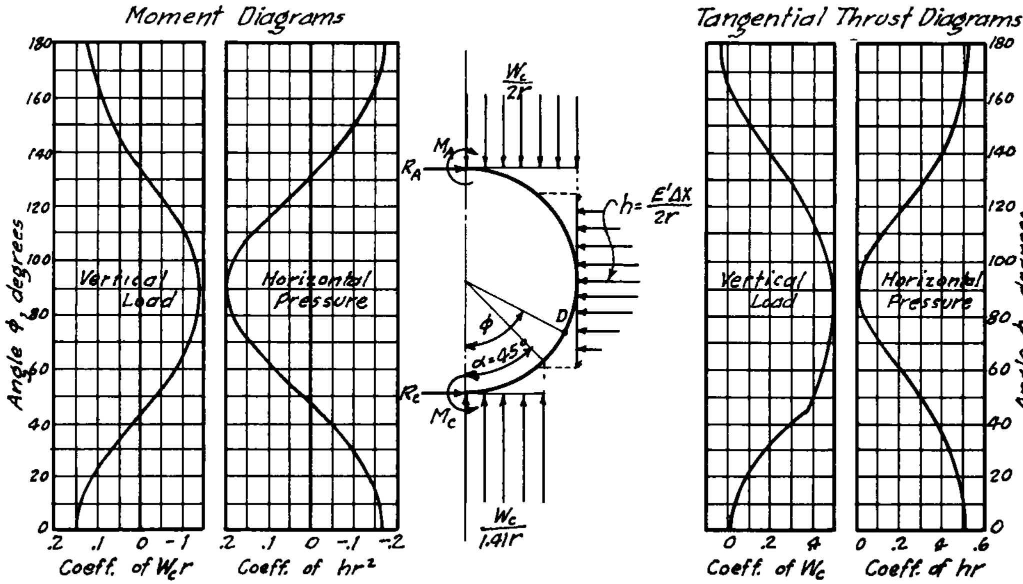


Figure 19. Moment and thrust diagrams, flexible pipe conduits.

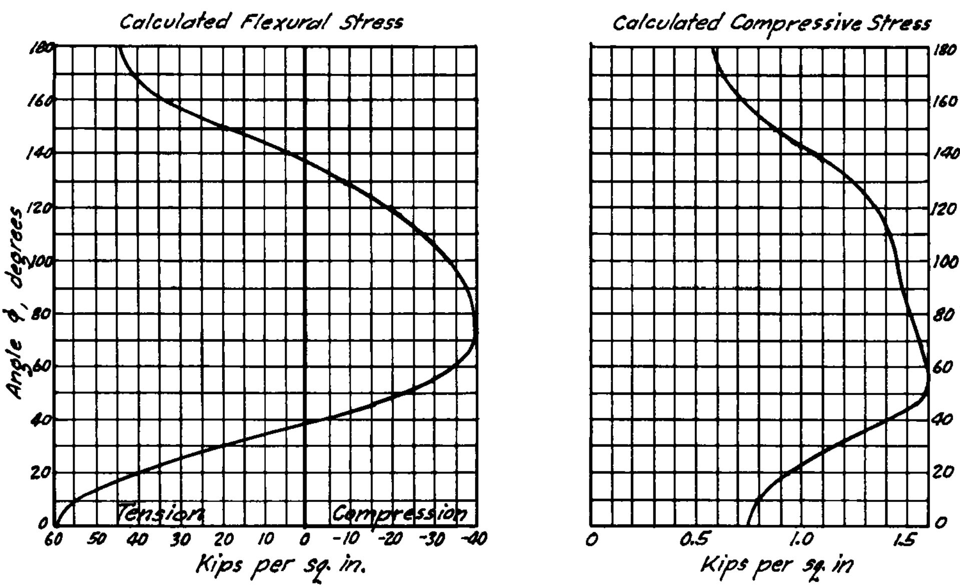


Figure 20. Calculated flexural and compressive stresses, experiment 1.

During the course of approximately 25 years of study of the flexible pipe problem, the writer has observed a number of situations that provide a background for design concepts in this field. One of these situations involves a 60-in. 10 ga. pipe in eastern Iowa, which was installed under about 9 ft of cover in the Fall of 1939. The pipe replaced a 16-ft span wood stringer bridge with pile and timber plank bulkheads as shown in Figure 21. When the pipe was installed, the timber deck was removed, but the bulkheads were left in place. The soil between the pipe and the bulkheads was pneumatically tamped up to the top of the pipe, but it consisted of top soil containing considerable organic matter and was not high-quality material.

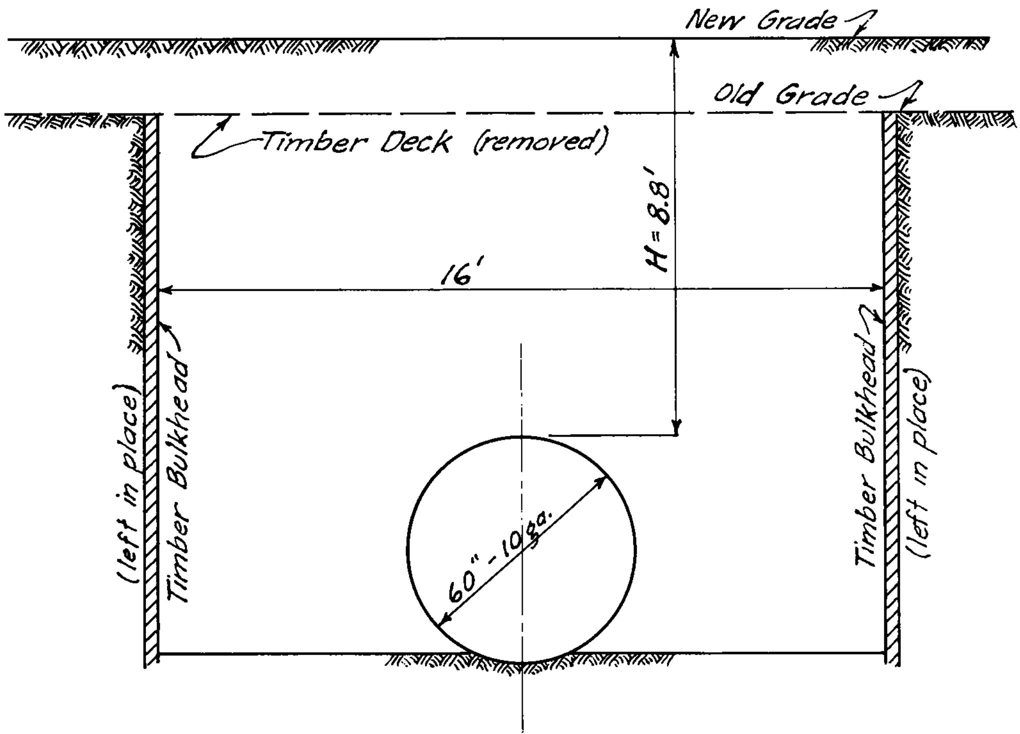


Figure 21. Culvert 1497, Linn County, Iowa.

The pipe was "shop strutted"; that is, horizontal tie rods and turnbuckles were installed on the horizontal diameter at 2-ft intervals throughout the length of the pipe. The turnbuckles were drawn up so that the initial vertical diameter before placement of sidefills was approximately 63.4 in. and the initial horizontal diameter was 58.2 in. Diameters were measured between points marked at the top, bottom and two sides at stations 10 ft apart throughout the length of the culvert. Repeat measurements of diameters were made at frequent intervals during the first 14 months after completion of the fill and at infrequent intervals since that time. The most recent measurements were made on September 26, 1961. Graphs of diameter changes at Sta. 0+30, which is under the center of the roadway, are shown in Figure 22. A photograph of the interior of the pipe at this same station is shown in Figure 23. One can practically "see" the bending moment in the pipe wall in this picture.

This case history is presented, not because it is a typical or usual situation. In fact, it is very unusual. It is presented to demonstrate and emphasize the fact that the pipe undergoes bending and deflection and in so doing must be able to develop sufficient passive resistance pressure in the sidefill soil to hold it in equilibrium, or deflection will be excessive.

One can only hypothesize as to the details of the cause of the unusually large deflection of this pipe. An hypothesis is that the timber plank bulkheads have probably rotted away through the years and permitted the sides of the pipe to move outward, even though columns of soil $5\frac{1}{2}$ ft wide intervened between the pipe and the bulkheads. How-

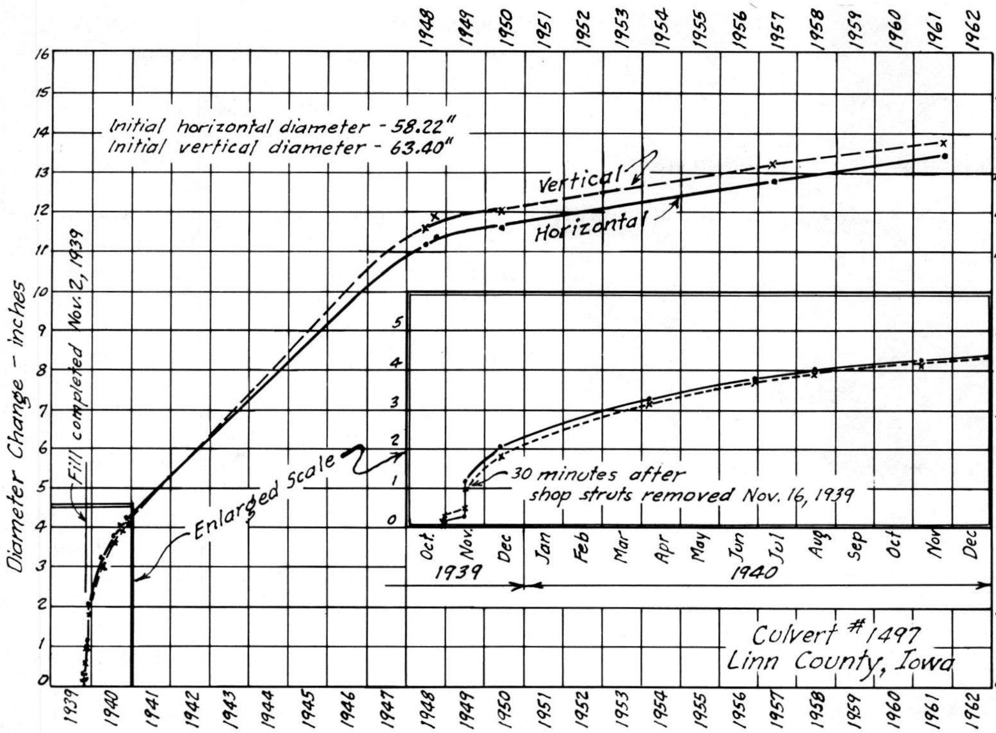


Figure 22. Culvert 1497, Linn County, Iowa.

ever, the sides of the pipe have moved outward more than the thickness of the timber planking, which was probably 3 in. and certainly not more than 4 in., whereas the horizontal diameter increased by a total of 13.4 in., or an average of 6.7 in. on each side. It appears that there was considerable compression strain in the sidefill soil columns in addition to the decay of the timber planking.

A 96-in. standard corrugated metal pipe which completely collapsed is shown in Figure 24. This pipe was also shop strutted. Unfortunately the tie rods were left in place during construction of the fill. The pipe was not permitted to deflect in a normal manner and it is believed this caused abnormally high stresses in the pipe ring and led to the collapse. Many of the tie rods were actually pulled in two in the region of the failure. An additional fact was that tests of the sidefill soil indicated that it was soft, yielding material, even when compacted. However, because the sides of the pipe were not permitted to move outward an appreciable amount against the soil, the poor quality of the material is not thought to have been a major factor in the failure.

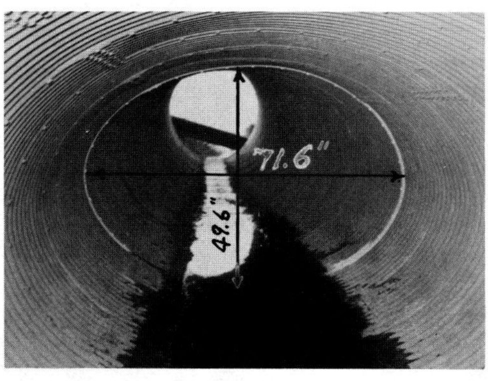


Figure 23. Excessive deflection of a 60-in. 10 ga. corrugated metal pipe culvert under 9 ft of fill.

Figure 25 is a view of a 144-in. structural plate pipe taken from inside looking toward one end. The pipe is on

Figure 24. Failure of 96-in. 8 ga. corrugated metal pipe.

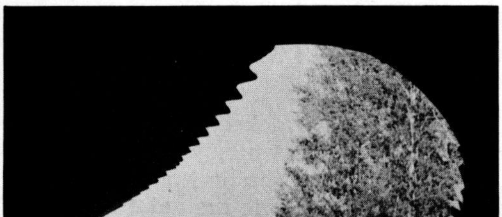


Figure 25. End view of 144-in. structural plate pipe on shrap skew.

a rather sharp skew, and the ends were beveled to conform with the side slopes of the embankment. As a result earth pressure acted against one side of the pipe while there was no compensating or balancing pressure on the opposite side and the inward bulging action resulted. This is not unusual when sharply skewed pipes are beveled at the ends in this manner. A more satisfactory treatment would be to carry the pipe beyond the side slope to a square end and then build a berm out from the embankment to envelop the pipe.

A partial seam failure in this same pipe is shown in Figure 26, and, again, one can "see" the bending moment in the pipe wall. This condition is thought to have been agravated by the fact that the plates in the bottom of the pipe were thicker than those in the sides and the crown. The seam shown is at the junction of the thick and thinner plates, and the sudden change in plate stiffness appears to have caused a concentration of bending moment stress with the result shown.

Another situation that came to light in connection with this structure is the fact that apparently the pipe was bedded on a flat surface without being shaped to fit the curvature of the bottom plates. The soil at the sides was tamped under the haunches of the pipe but did not fill the triangular wedge-shaped space between the pipe and bedding, as shown in Figure 27. It was possible to identify this unfilled area by tapping on the bottom of the pipe with a ball peen hammer. The open space appeared to be from 12 to 18 in. wide on each side at the bottom.

The failure to fill the space undoubtedly caused the bottom reaction to be concentrated on a relatively narrow longitudinal element of the pipe, resulting in greater deflection and bending moments than there would have been if the bedding had been shaped to fit the pipe. Also, it is possible that this open space may provide a passageway for water outside the pipe in time of heavy flow and could lead to erosion and failure of the embankment.

Figure 28 shows line diagrams of two corrugated metal pipes in the loess region of western Iowa which have settled an excessive amount due to compression strain in the underlying soil under the weight of the overlying embankment. These pipes were constructed on a flat uniform grade from inlet to outlet. The maximum sag

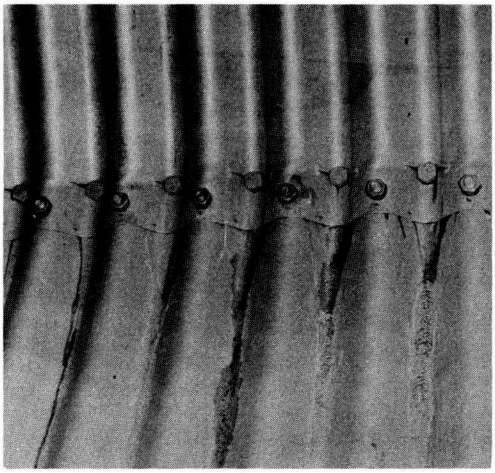


Figure 26. Partial seam failure in 144-in. structural plate pipe.

in the flow line of these culverts amounts to 33 to 40 percent of the pipe diameter. This has resulted in extensive accumulation of water and silt inside the pipes and is generally unsatisfactory.

The analytical and experimental studies referred to plus observation of the performance of some actual structures in the field, lead the writer to offer the following series of "Do's and Dont's" in the design and construction of flexible pipe culverts:

1. Investigate the probable deflection of the pipe and hold it within safe limits. Do not rely solely on the tangential thrust in the pipe wall as a basis for design.

2. Remember that a flexible pipe fails primarily by excessive deflection rather

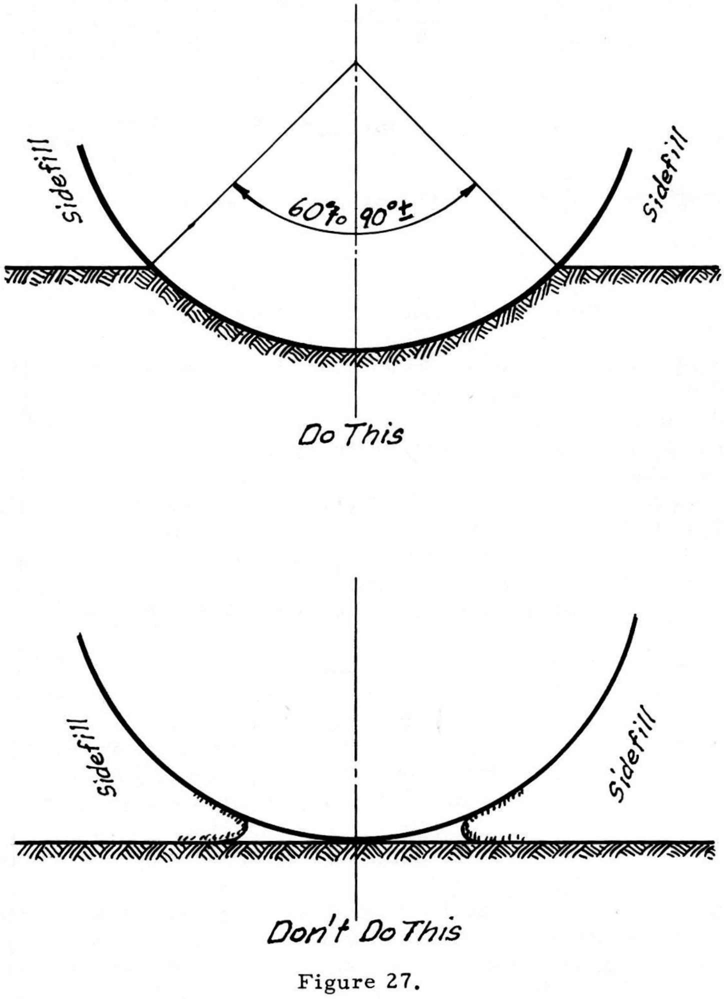
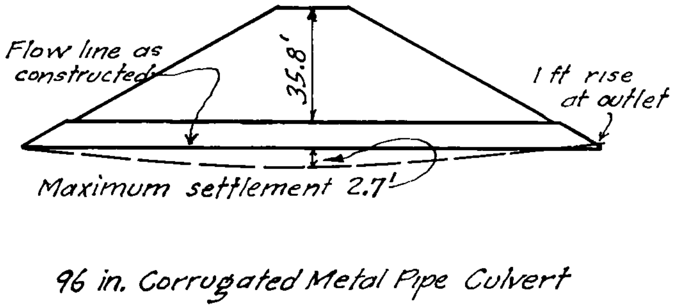


Figure 27.



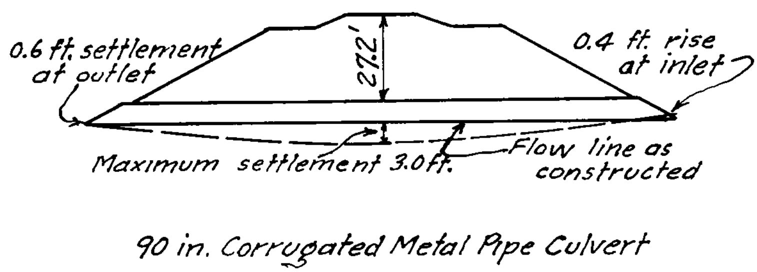


Figure 28. Settlement of two flexible pipe culverts in loess region of western Iowa.

than by rupture of the pipe wall. Do not design the pipe on the basis of maximum bending stress.

3. Watch a pipe which has been pre-deformed by timber struts or tie-rods during construction of the overlying embankment. Be prepared to remove them at once if the pipe fails to deform in a normal manner or shows signs of distress and local bending. Do not leave pre-deforming devices in place very long after pipe has been backfilled and fill has been well started.

4. Select good compactible soil to be placed at the sides of the pipe. Do not use just any old "dirt" which is most readily available for the sidefill soil.

5. Compact the sidefill soil by acceptable controlled methods for a distance on each side of at least two pipe diameters; more if feasible. Do not merely run a tractor back and forth parallel to the pipe a few times and consider that the soil has been well compacted.

6. Place and compact the sidefill soil evenly on both sides of the pipe throughout its whole length. Do not let the fill on one side of the pipe get to be more than 6 in. higher than the fill on the other side at any time.

7. When a flexible pipe drainage structure consists of multiple parallel lines, space the pipes far enough apart to permit hauling and compacting equipment to operate between them. Do not space the pipes so close together that it is difficult to place and compact the sidefill soil properly.

8. Shape the soil bedding to fit the bottom of the pipe for approximately 60° to 90°. Do not lay the pipe on a flat bed of soil.

9. Change plate thickness gradually, if plates of various gages are used. Do not bolt thin plates directly to thick plates, especially in regions of high bending moment.

10. Carry skewed pipes out a sufficient distance beyond the embankment slope to per-

mit the end bevel to be at right angles to the pipe barrel. Do not bevel a skewed pipe parallel to the embankment slope.

11. Estimate the amount of settlement of the natural ground surface under an embankment and camber the invert grade of the pipe culvert a sufficient amount to insure against a sag or backslope in the line. Do not construct the pipe on a uniform grade from inlet to outlet in regions where substantial subsidence of the supporting soil may be expected.

12. Remember that flexible pipes usually continued to deflect long after the embankment has been completed. Do not assume that the pipe deflection at the time the embankment is completed is the maximum deflection that will develop.

REFERENCE

9. Watkins, R. K., and Spangler, M. G., "Some Characteristics of the Modulus of Passive Resistance of Soil: A Study of Similitude." HRB Proc., 37:576 (1958).

H. L. WHITE, Chief Sales Engineer, Metal Products Division, Armco Steel Corporation, Middletown, Ohio—Much research has gone into both laboratory and field experimentation in respect to flexible metal culverts in the past five years by Metal Products Division of Armco Steel Corporation. (This organization until the first of the year was known as Armco Drainage & Metal Products, Inc., a subsidiary of Armco Steel Corporation.)

One fact has been well proven by these most recent experiments and actual jobs—flexible culverts properly backfilled do not deflect much more than 1 percent and usually less than 2 percent when the fill load is placed on them. By proper regulation of the compaction procedure during backfilling, the structure can be elongated vertically from its assembled size approximately 2 percent before the fill load is placed on it. On the placement of the fill load above it, the structure will then return to approximately its fabricated size.

The recent installation of a 10 percent vertically-ellipsed 24-ft diameter structural plate pipe in the DeCoursey Yards of the L&N Railroad south of Cincinnati, demonstrated this most dramatically. This structure and some 25 others ranging in size from 15 to 21 ft have been installed in the past few years by proper backfilling and adequate compaction methods, using material of good quality, and each of them has proven this point.

The installation at Cullman, Ala., of three 7-ft diameter tubes, each 512 ft long, under 134 ft of cover, was one of the first installations to show that properly installed flexible culverts act primarily as compression rings. The greatest deflection observed in these structures under the 134 ft of fill was 2 percent from their 3 percent vertical elongation. A technical report on these structures was presented to the Highway Research Board at the 1952 annual meeting.

This then is the hypothesis on which Armco Drainage & Metal Products issued the method of calculating these structures now known as the "ring compression method" (9). It is well recognized that this is a method that predicates that the pipe be subjected to little or no bending stress. It is also well recognized that to take full advantage of this particular method of calculating the necessary strength of the structure, quality and compaction of the backfill must be such as to maintain the structure very close to its fabricated shape.

The work done by Mr. Koepf in investigating the action of culverts of aluminum material is very excellent and thoroughly instrumentated and well analyzed. However, it should be recognized that the trend is away from obtaining compaction at only 85 Proctor on the backfill material. Though it is true that this compaction can be obtained under careless circumstances, or what many people have been led to believe is average installation practice, it has definitely been proven that higher compaction is usually obtained.

In one installation of twin 14-ft diameter sectional plate pipe (not Armco pipe) on Ohio 674, two miles south of Canal Winchester, field compaction tests were taken on each lift of the backfill. Compaction was attained by the use of both pneumatic and vibratory compactors. The average compaction was 97 percent AASHO. The structure was installed in March and April 1955, and no progressive deflections have occurred.

More and more engineers and contractors are becoming aware of and are using the method of well-compacted backfill to maintain flexible structures with very minor vertical deflections under full loads. For this reason, the author believes that the work done by Mr. Koepf should come under the heading of "Investigation of the Action of Flexible Culverts Having a Compacted Backfill of 85 Percent Proctor" and should not be considered as covering the entire range of the possibilities of installation of such culverts.

Professor Spangler's remarks in respect to the hypothesis made 25 years ago for the magnitude and distribution of loads and pressures on flexible pipes under earth cover are as true now as they were then—they are still approximate assumptions made for the convenience of mathematically analyzing a pipe based on these assumptions. They may or may not pertain. It is rather doubtful if a pressure diagram such as has been approximated by this hypothesis could have been obtained on the Lafleche, Saskatchewan, culvert. This was a 72-in. diameter culvert in a $\frac{1}{2}$- by $2\frac{2}{3}$-in. corrugation ellipsed vertically during installation some 3 percent and failed in a compression type of buckle by 34 ft of cover in a dam installation. The gage of this particular structure was 20—approximately as thick as a well-worn dime—and it is doubtful if the moment strength of this culvert could alter a pressure diagram of the surrounding earth which had to be approximately uniform radially to maintain this culvert after it had taken the 2 percent deflection under cover.

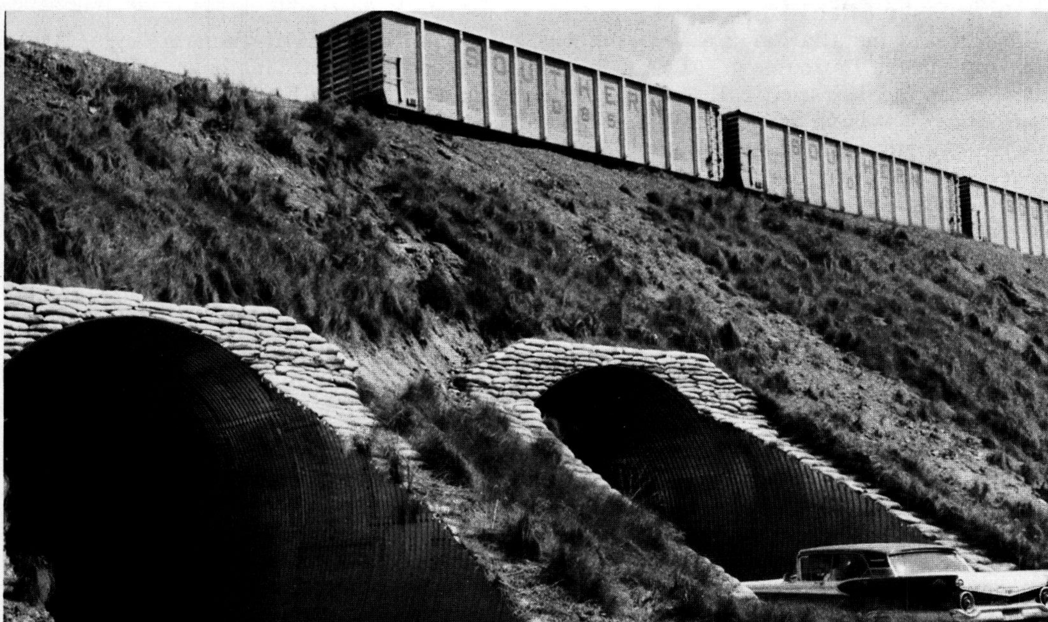


Figure 29. Twin 17 1/2-ft diameter, 7 gage MULTI-PLATE Pipes carrying highway under industrial railroad tracks at Parrish, Ala. Fill is 25 ft high.

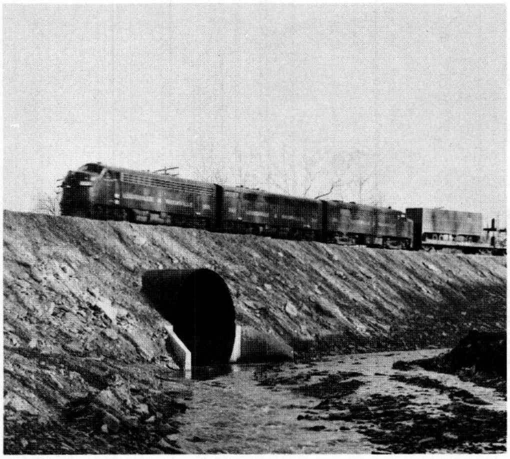


Figure 30. A 20-ft diameter, 1 gage MULTI-PLATE culvert under mainline of L&N RR near Independence, Ky.

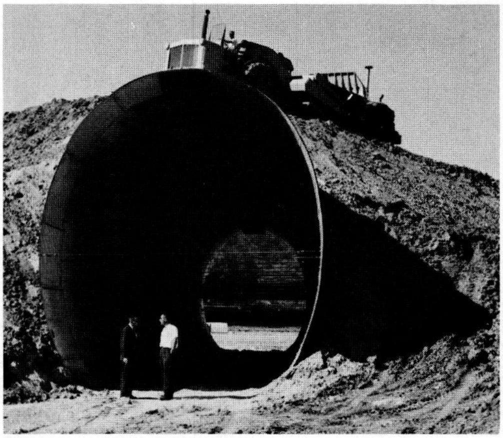


Figure 31. A 24-ft diameter, 10 percent vertically elongated structure to serve as locomotive underpass under hump at new classification yard, DeCoursey, Ky. Structure, after dead load had been applied, came to 1/2 in. of designed diamter.

To cite another example, an 18-ft diameter 5 percent vertically elongated structural plate conduit of 10 gage steel was placed in Greene County, Ohio, and so compacted that the 9-ft height of cover deadload still allowed the culvert to remain approximately 1 in. above the vertical formed dimension. Again, the moment strength of this structure is very slight as compared to the compression strength, and because the culvert remained in practically the shape in which it was fabricated that none of this small amount of moment strength was used. It is the writer's opinion that there is much more to be gained economically by operating on the backfill to obtain proper passive resistance and proper shape of culvert than there is to operate on the culvert itself in terms of moment strength.

Sufficient moment strength is required in such culverts to enable them to be handled, be backfilled, and contain adequate provisions against buckling of the wall under compressive stress. The best-known data in existence for how much this needs to be is contained in the presently published tables for gages of flexible steel culverts. Utilizing these gage tables as the accumulated data from many thousands of installations,

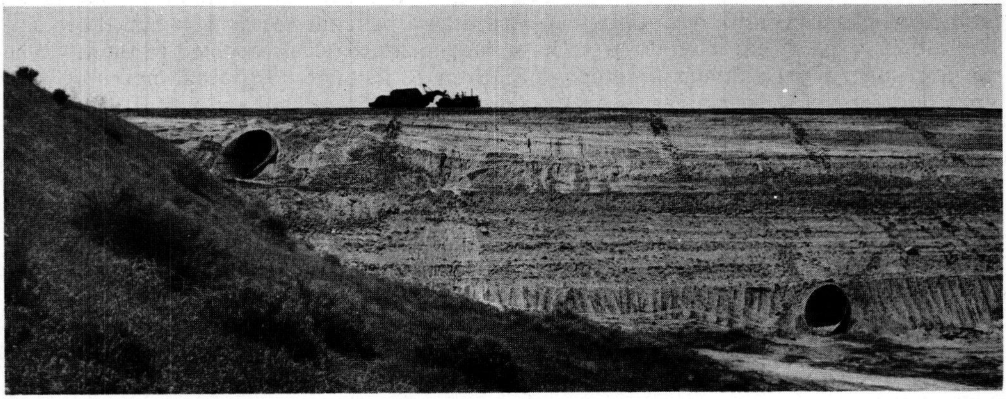


Figure 32. Upper structure is 18-ft diameter, 8 gage MULTI-PLATE under 17 ft of cover; lower one is 10-ft diameter, 1 gage under 70 ft of cover; located at Ice Harbor Dam, Wash.

Figure 33. Triple 20- by 10-ft MULTI-PLATE Arch located under Interstate 90 in Cleveland, Ohio. Maximum fill on this 1,000-ft long culvert is 20 ft, yet deflection is practically negligible.

criteria can be and have been established for the flexibility factor of such structures. These criteria are available from Metal Products Division of Armco Steel Corporation in Middletown, Ohio. They have been published in papers presented to engineers throughout the country.

Adequate safety factors are employed in the ring compression design of flexible structures to provide safety against column buckling until this particular point can be brought to a finer mathematical conclusion than now exists. Work is being carried on in university laboratories to determine the relationship between the quality of the backfill and the required column properties of the culvert wall.

In conclusion, the writer would like to submit that there are many more culverts in existence performing with deflections of less than 2 percent vertically than there are of those that were so unfortunate as to have inadequate backfill applied to them. The ratio is probably many thousand to one. Figures 29 through 33 are examples of a few of such culverts in larger sizes and various shapes. The design and installation of flexible pipe culverts in these figures has progressed to a very conclusive point in just the past few years although no material available in today's text books adequately describes the science.

R. L. BROCKENBROUGH and J. ALAN MYERS, United States Steel Corporation, Pittsburgh, Pa.—Mr Koepf's paper is of interest to those associated with the design of flexible underground conduits because it presents a summary of the results of a test made on corrugated aluminum alloy pipe under a 30-ft embankment. The test has contributed to expanding the limited knowledge and experience associated with the use of corrugated aluminum pipe for the drainage application. On the basis of an evaluation of the data collected, the author presents a table of recommended maximum fill heights up to 50 ft for certain combinations of sheet thickness and pipe diameter. The writers believe several of the assumptions the author has made in evaluating the collected test data require critical examination.

Figure 7 shows the distribution of earth pressures across the top of a test culvert under 30 ft of earth fill. The vertical earth pressure directly over the center of the pipe is only about one-fourth the vertical earth pressure measured in the regions adjacent to the sides of the pipe. (This loading is similar to a "ditch condition" loading.) Therefore, as mentioned in the paper, the actual pressure on the culvert is considerably below the pressure that would be predicted by assuming a settlement ratio of zero. (The settlement ratio is an abstract quantity which, when combined with other terms, may be used to calculate the load on a culvert. When the settlement ratio is zero, the load on the culvert is equal to the total weight of the column of earth directly over it.) The loading condition attained in the test leads the author to state:

> Normal design tables for aluminum culvert herein are then based on the settlement ratio of zero in conformance with standard practice on other materials. Actually, such a decision builds into the aluminum culvert a safety margin somewhat higher than for other materials due to the strong tendency toward ditch condition loading. This margin has been considered in later selection and evaluation of safety factors on pipe stress.

The writers cannot agree with this statement because it implies ditch condition loading is a particular characteristic of corrugated aluminum alloy culverts. The strong tendency toward the ditch condition loading experienced in the test probably resulted from the type of soil that was used and the construction procedure followed in making the embankment. The pressure distribution noted probably would have occurred for a corrugated steel culvert or any other flexible underground conduit under the same installation conditions. The embankment, if made from another type of soil, might not have undergone ditching action to the same extent. Consequently, it is erroneous to assume there is a higher safety margin inherent in the use of aluminum alloy culverts.

The pressure distribution noted is important, however, because it once again confirms that the tendency toward ditch condition loading is a characteristic of all conduits flexible in nature, regardless of the kind of material from which the structure walls are made.

The author used a safety factor of 2.50 against yielding in determining the allowable design stress for combined bending and axial compression of the aluminum culvert material. The resulting maximum allowable stress was used in developing the recommended fill heights from the strain data collected. The use of the 2.50 safety factor was based on the "ASCE Specification for Structures of Aluminum Alloy 6061-T6." Although this factor may be adequate for ordinary aluminum engineering structures, experience gained from the use of other materials indicated a higher factor of safety should be used for a flexible metal culvert. This is true because exact loadings on underground conduits are difficult to determine. Soft spots in an embankment or undermining of a culvert can cause unpredictably high bending moments with resulting stresses considerably above the stresses that might be ordinarily encountered. The increased tendency toward the ditching condition claimed for corrugated aluminum alloy culverts should not give confidence to the choice of the safety factor, because the writers believe the claim is not necessarily valid for reasons already mentioned.

Actually, an over-all factor of safety of 2.50 was not adequately demonstrated. In the tests performed on the full circle aluminum alloy culverts, the 30-ft embankment loading produced average measured vertical pressures from 1.0 to 1.4 times the pressures that might be encountered under the recommended fill heights for other soil and field installation conditions. For example, in Table 1, the author recommends a maximum fill height of 14 ft for a 60-in. full circle aluminum culvert. A design earth pressure of about 10 psi (assuming a settlement ratio of 0- and 100-pcf material) would ordinarily be used for this height. The mean earth pressure above the 60-in. test culvert was 14.4 psi at maximum test load. To demonstrate experimentally that this culvert actually had a safety factor of 2.50 against failure, it would have been necessary to load it to a fill height that would develop a measured pressure of 25 psi (2.5 times the assumed design pressure of 10 psi). The author implies that the safety factor exists because the proposed table has been designed so that the maximum stress developed by a pressure 2.5 times the usual design pressure (presumably 25 psi for the 60-in. diameter culvert) is approximately equal to the material yield stress. This would only be true, of course, if it could be safely assumed that ring buckling does not occur before yielding of the wall material. The validity of this assumption was not established theoretically in the paper nor was it demonstrated experimentally by the test.

The paper makes only slight mention of ring buckling as a structural design consideration for flexible metal pipe. It appears no account has been taken of this phenomenon in establishing the recommended fill heights shown in the proposed gage vs fill height table. The writers believe buckling cannot be overlooked in designing an aluminum alloy culvert. Reynold K. Watkins (10) describes a study he conducted in an attempt to account for the possible influence of ring stiffness and the effect of elastic deformation on the load that will cause ring buckling. The results of the study led Watkins to conclude: "Buckling of the ring is a critical consideration in the design of flexible conduits." In other words, for certain combinations of soil type, loading, diameter, and wall stiffness, a flexible conduit may fail due to elastic instability of the wall, rather than crushing of the joint, excessive deformation, or yielding of the wall material. Buckling will become increasingly significant as one tries to refine existing flexible metal culvert designs to make more efficient use of materials. Because of aluminum's relatively low modulus of elasticity, ring buckling (which is dependent on wall stiffness, EI) possibly will be, in many cases, the critical consideration in the structural design of corrugated aluminum alloy culvert.

REFERENCE

10. Watkins, R.K., "Failure Conditions of Flexible Culverts Embedded in Soil." HRB Proc., 39:361-371 (1960).

A. H. KOEPF, Closure—Discussions by M. G. Spangler, H. L. White, R. L. Brockenbrough, and J. Alan Myers, have done an excellent job of further expanding the concepts by which flexible metal culverts might be evaluated. Emphasis is placed on the ability of flexible culvert to reduce its loading by deflection, thus extending the limits of design in the higher fills. Mr. White's indication that higher fills can be attained when the soil is well compacted under controlled conditions is well supported. In this instance, bending stress stays relatively low compared to joint strength. It was the intent of this study to establish a conservative floor under design limits and consider the effect of many potential methods of failure, selecting for a design limit that method of most importance for each size of pipe and gage. Certainly where conditions prevail which are better than the base and can be constantly substantiated in the field, the design values can be reviewed and design limits revised.

In the instance of the discussion by Mr. Brockenbrough and Mr. Myers, it should be pointed out that the selection of 30 ft as the maximum fill height was considered sufficient to demonstrate the pattern of strain, pressure, and deflection; and was not selected to demonstrate a condition of imminent failure. It is felt that if the former follows a rational behavior, the latter can be demonstrated by extrapolation of data.

M. G. SPANGLER, Closure—Mr. White and the writer are in complete agreement on the proposition that high-quality backfilling material, properly placed and compacted around a flexible pipe culvert, will produce an installation that is highly satisfactory from a structural standpoint. We disagree concerning design methods for taking into account the role that the backfill material plays in establishing the structural equilibrium of the flexible pipe.

Mr. White approaches the design of a flexible pipe culvert by assuming that the soil around the structure is of such high quality that deflection of the pipe can be ignored in the design procedure. He has demonstrated this idea by means of a lightweight tin cylinder confinded between two wooden blocks. These blocks inhibit and minimize the deflection of the thin ring so that it will carry a relatively tremendous load. He has described this wooden block support as a "perfect soil" and this description is very appropriate.

Unfortunately only a small percentage of actual soil backfills, placed and compacted by human beings, can be said to approach this state of perfection, although some do. For example, the Cullman County, Ala., installation referred to, has a near-perfect backfill environment and the deflection of the 84-in. pipes has been negligible. On the other hand, there are numerous installations wherein the soil backfill has not been of such high quality, and pipe deflection has been a factor.

In contrast to this philosophy, the writer believes that the quality of a soil backfill can be expressed quantitatively and that this quantitatively expressed quality has a direct bearing on the required stiffness of the pipe ring and on the pipe deflection that will develop during its functional life.

Mr. White's concluding paragraph in which he states that the ratio of culverts in service with deflections less than 2 percent to those with greater deflections is probably "many thousand to one," is far two optimistic. In 1943 a leading flexible pipe manufacturer conducted a survey of culvert pipe deflections which embraced 239 run-of-the-mine structures in widely dispersed geographical locations. These pipes were all of the "structural plate" type and ranged in size from 60 to 180 in. The heights of fill cover ranged from 0 to 90 ft.

The deflection of one of these culverts was described as "excessive—not used." The deflections of the remaining 238 culverts ranged from -4.97 to +12.10 percent of the nominal diameter and the average deflection was +2.32 percent. It is believed that those pipes with negative and very small positive deflections were probably strutted or fabricated to an initial out-of-roundness, but specific information on this matter is not available.

Using the 2 percent break-off point it is noted that 131 pipes deflected less than 2 percent of the nominal diameter, whereas 108 deflected 2 percent or more. The ratio is 1.21 to 1. Granted that installation know-how and procedures have improved during the past 20 years, it is doubtful if the ratio of "many thousand to one" is valid.

THE NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL is a private, nonprofit organization of scientists, dedicated to the furtherance of science and to its use for the general welfare. The ACADEMY itself was established in 1863 under a congressional charter signed by President Lincoln. Empowered to provide for all activities appropriate to academies of science, it was also required by its charter to act as an adviser to the federal government in scientific matters. This provision accounts for the close ties that have always existed between the ACADEMY and the government, although the ACADEMY is not a governmental agency.

The NATIONAL RESEARCH COUNCIL was established by the ACADEMY in 1916, at the request of President Wilson, to enable scientists generally to associate their efforts with those of the limited membership of the ACADEMY in service to the nation, to society, and to science at home and abroad. Members of the NATIONAL RESEARCH COUNCIL receive their appointments from the president of the ACADEMY. They include representatives nominated by the major scientific and technical societies, representatives of the federal government, and a number of members at large. In addition, several thousand scientists and engineers take part in the activities of the research council through membership on its various boards and committees.

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The HIGHWAY RESEARCH BOARD was organized November 11, 1920, as an agency of the Division of Engineering and Industrial Research, one of the eight functional divisions of the NATIONAL RESEARCH COUNCIL. The BOARD is a cooperative organization of the highway technologists of America operating under the auspices of the ACADEMY–COUNCIL and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the BOARD are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.