Uplift Failures of Corrugated Metal Pipe

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Surveys of county engineers in Iowa revealed that between 1970 and 1975 and between 1983 and 1988 corrugated metal pipe (CMP) uplift failures occurred at a rate of six per year. A similar survey of North American Departments of Transportation (DOTs) revealed only nine failures between 1987 and 1992. All failures involved pipes with diameters greater than 1.83 m. Fifty percent of the agencies responding to the second survey indicated they had standards for end restraint. About 6 percent of the agencies avoid uplift problems by not using corrugated metal pipe and about 12 percent of the agencies limit the diameters of CMP. Eight agencies provided data that allowed calculation of end restraint force as a function of pipe diameter. The forces provided by the end restraints range from about 50 to 300 kN for a pipe 2 m in diameter. A simplistic and very conservative analysis that treats the CMP as a beam was conducted as a basis for comparison. Six standards recommend forces lower than the analysis whereas three agencies actually recommend resisting forces greater than that provided by the analysis. Two CMPs that failed in uplift were analyzed and the apparent force that caused failure was back calculated. For one pipe that had no tie-down, the calculated failure force was below all of the standards; however the presence of a cutoff indicates that cutoffs by themselves are not effective countermeasures to uplift. In the second case history where end restraint was provided, the calculated failure force was greater than that required by six of the nine agency specifications.

Corrugated metal pipe (CMP) culverts are important components of the drainage systems of the country's transportation system. In Iowa, many county engineers have used large diameter flexible pipe culverts to replace small bridges and have realized significant savings; however, in some situations, CMPs have failed as a result of longitudinal uplift. The objectives of this study are to (1) determine the scope of the problem within Iowa and in North America, (2) identify a unique set of pipe configurations that might be more conducive to uplift, and (3) identify types of tie-downs currently being used to resist uplift and synthesize the resisting forces provided by the structures.

PREVIOUS STUDIES

Only one publication was found that documented uplift failures of CMP (*1*). In that report, buoyancy was assumed to be the cause the uplift with no consideration given to the dissipation of pore pressures through seepage. The analysis for a 2.44-m diameter pipe beneath a soil slope of 2.5 to 1 resulted in a moment of 404 kN.m.

The FHWA issued Notice N 5040.3 dated April 26, 1974 (D.C. Coy, personal communication). This notice recommended all pipe culverts with diameters 1.22 m and larger be provided with end protection and was accompanied by design standards for culverts up to 4.57 m in diameter.

The first assessment of CMP uplift problems in Iowa occurred in 1975 when the Iowa Department of Transportation (DOT) surveyed county engineers requesting information on uplift problems within the previous 5-year period (*2*). The questionnaire made no distinction between flotation and folding failures of beveled or step beveled ends. Fifty of Iowa's 99 counties responded and the responses are shown in Table 1. Thirty failures occurred in 8 counties with only 5 percent of pipes with diameters less than 2.43 m diameter having problems; however 16 to 18 percent of pipes with diameters greater than 3.07 m had problems. These data motivated the Iowa DOT in February 1976 to issue a letter to all county engineers urging them to anchor or reinforce inlet ends of unprotected flexible pipe.

Not withstanding these warnings from FHWA and Iowa DOT, CMP uplift failures continued to be reported in Iowa. Concern that current design and/or construction practices are inadequate lead to this study.

SURVEYS TO DETERMINE THE SCOPE OF CMP UPLIFT PROBLEMS

Survey of Iowa Counties 1983 to 1988

A survey of Iowa county engineers in 1988 revealed that 31 CMP had failed by uplift in the previous five years. It was hypothesized that it might be possible to identify a unique set of conditions that resulted in these uplifts. For example, are uplift failures more frequent in skewed and/or projecting conduits where less favorable hydraulic conditions exist at the inlet?

These data indicate that 12 percent of 68 counties who responded to the questionnaire reported CMP failures. This compares to 16 percent of responding counties in the 1975 survey. Table 2 summarizes the range of culvert sizes involved in uplift failures. All uplift failures were associated with pipes 1.83 m in diameter or more. In one instance, the pipe uplift and subsequent washout caused the death of a motorist.

Table 2 shows the range of culvert sizes involved in the failures and lists less than 31 events because some sizes had more than one failure. The majority of respondents indicated that plugging or partial blocking of inlets by vegetative debris contributed to the uplift failures. Tables 1 and 2 suggest that projecting CMPs are not more likely to experience uplift difficulties. No unique geotechnical or hydrologic conditions were identified as contributing to the failures; however, the problems appear more common in regions of the state where significant elevation drops exist across the culverts. This suggests that the flow through the CMP was inlet-controlled and that in part of the pipe the flow will be shallow, high velocity (supercritical). Under these conditions, with the pipe only partially full, blockage of the inlet would not be necessary because buoyant pore water forces outside the pipe could be greater than the resisting weight of the water in the pipe.

Of the counties reporting failures, 75 percent indicated that they used some form of tie-down including: piles and cables, concrete curtain walls, concrete slope collars, and sheet piling cut-off walls.

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TABLE 1 Summary of CMP Uplift Failures in Iowa 1970 to 1975

Pipe diameter (m)	Number of structures Projecting	Beveled	Number of failures Projecting	Beveled
1.52 to 2.44	226	166	2	11
2.46 to 3.05	19	46	1	5
>3.07	11	53	2	9

TABLE 2 Summary of CMP Uplift Failures in Iowa 1983 to 1988

Diameter (m)	length (m)	Inlet geometry*
1.83	76.8	unknown
1.98	32.9	beveled
2.29	36.6	projecting
2.59	16.5	beveled
2.74	18.9	unknown
2.74	21.3	beveled
3.25 x 2.11**	38.1	beveled
3.35	44.5	projecting
3.51	36.6	beveled
3.66	21.9	projecting
4.54 x 2.92**	46.3	beveled
4.52 x 2.92**	unknown	beveled
4.57	36.5	unknown
5.18	36.5	unknown
9.78 x 5.85**	79.2	unknown

** elliptical arch pipe
*Projecting inlets are CMP with square ends; beveled inlets are CMP with ends parallel to the embankment.

Survey of North America 1987 to 1992

Uplift problems

In order to define the uplift problem on a wider geographic scale and to identify the types of end restraints being used, Iowa DOT with Iowa State University sent questionnaires to the DOTs in each of the 50 states, Washington, D.C., Puerto Rico, and the eight provinces of Canada, requesting information on the use of restraints and any uplift problems encountered between 1987 and 1992.

Fifty-two of 60 agencies queried responded to the questionnaires. Of those responding, 9 (17 percent) agencies reported uplift problems in the previous 5 years, and 26 of the 52 report incorporating some type of an uplift restraint. Eighteen of those 26 agencies developed the restraints in response to earlier problems. Table 3 summarizes data from seven of the reported uplift problems. Two agencies that experienced uplift problems provided no specific data on the nature of their problems. In all cases, except one, the pipes were circular with diameters ranging from 0.91 to 2.90 m. For the agencies who reported soil cover depths, the cover ranged from 1.5 to 3 m with the deepest cover of 3.05 m over the largest diameter pipe (2.90 m) reported by Agency 6. All problematic pipes had projecting inlets except for one step beveled inlet and one beveled inlet. Table 3 suggests that skew is not an essential contributor to uplift. In all cases, the damaged pipes were replaced with new CMP and in most situations end restraint was added.

TABLE 3 Summary of CMP Uplift Failures, United States and Canada 1987 to 1992

Agency	Diameter or span/rise (m)	Length (m)	Skew (deg)	Cover depth (m)
1	4.5/2.7	nd	nd	nd
2	1.82	nd	90	1.52
	2.44	nd	90	2.44
3	1.52	5.8	nd	nd
4	0.91	12.2	10	(very little)
5	1.52	nd	nd	1.52
6	2.90	50	30	3.04
7	2.44	27.4	0	1.83

"nd" indicates that no data are given.

Types of End Restraints

Twenty-two agencies provided copies of their design standards for end restraints. The variety of end restraints can be classified as anchors, head walls, wing walls, and slope collars. Figure 1 shows schematic drawings of each type of end restraint.

Anchors consist of vertical concrete walls with considerable mass of concrete below ground, perpendicular to the axis of the pipe, that extend to midheight of the culvert. Bolts connect the concrete to the pipe. The pipe ends are beveled above the top of the concrete. In some situations, cutoff walls extend below the concrete anchors.

Head walls are vertical concrete walls, perpendicular to the axis of the pipe, that extend above the top of square ended pipe. Wing walls are similar to head walls but incorporate vertical walls on both sides at an angle to the axis of the pipe. The angled wing walls serve to direct flow into the pipe, prevent erosion or piping adjacent to the inlet, and add mass to resist uplift.

Slope collars may be either concrete or metal. The collars surround the culvert inlet, perpendicular to the pipe axis, and are parallel to soil slope of the embankment above the culvert.

Three agencies avoid CMP uplift problems by not using CMP. Six other agencies limit the maximum diameter of CMP, with the maximum diameters ranging from 1.37 to 2.13 m, thereby reducing the probability of uplift failure.

Anchor walls are used by eight agencies, headwalls by six, wing walls by four, concrete slope collars by five, and metal slope collars by three. One agency uses anchor walls for CMP less than 1.22 m in diameter and either slope collars or wing walls for pipe larger than 1.22 m in diameter. A northern agency uses anchor walls on pipes 0.30 to 1.37 m in diameter with the latter as the maximum diameter CMP they will use. An agency from eastern United States recommends wing walls on CMP between 0.91 and 1.83 m diameter and headwalls as an option on pipes less than 1.22 m in diameter. The maximum diameter CMP that this state uses is 1.83 m. One north-central agency uses a system of longitudinal stiffeners. The variety of end restraints used suggests that in many cases the standards are based on experience and not on theoretical analyses or results of load tests.

Analysis of Resistance to Uplift

In order to compare the resisting forces of the various end restraints, an analysis was conducted in which the pipe was treated as a simple beam. The details of the analysis can be found elsewhere (*3*) but

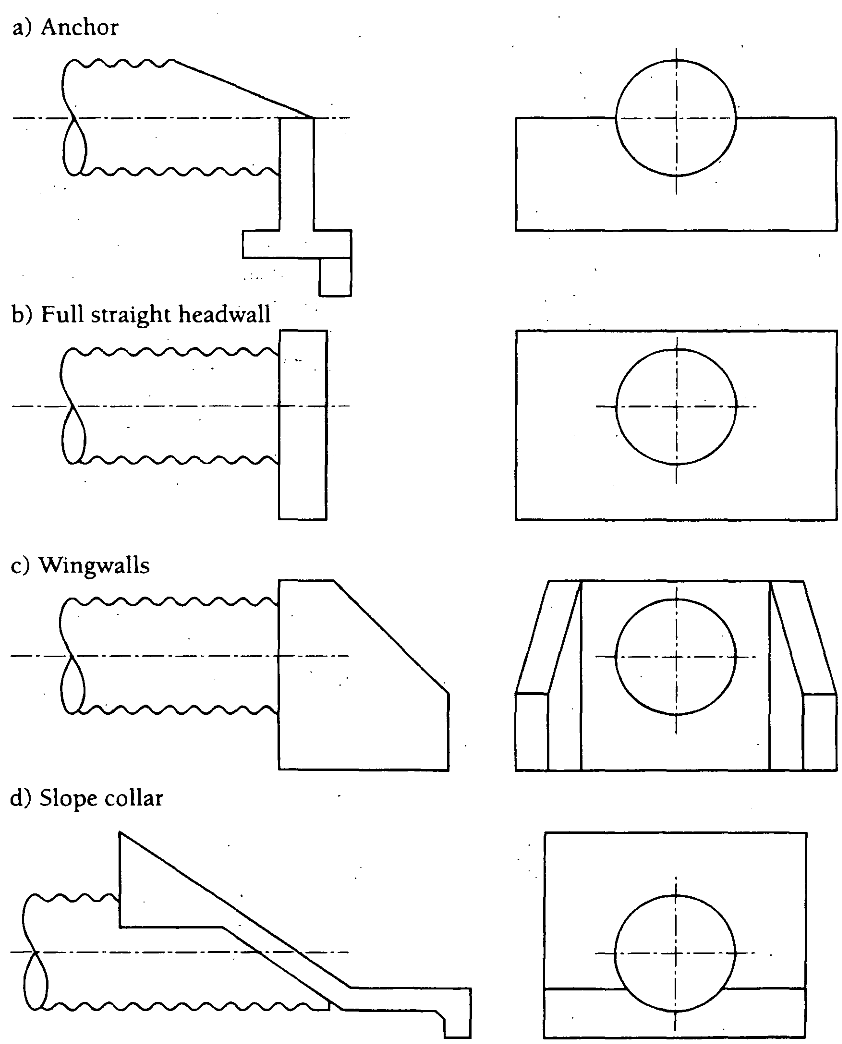

FIGURE 1 Types of headwalls described by agencies responding to survey.

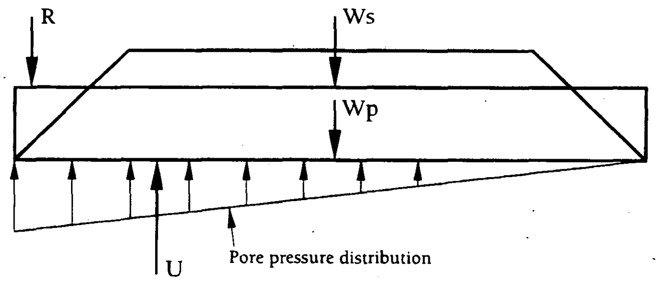

FIGURE 2 Schematic of CMP showing pore water pressure distribution and resultant forces for simplified analysis.

an overview of the results is summarized here. The uplift was assumed to result from a linear distribution of pore pressure with the maximum head equal to the pipe diameter at the inlet and a minimum head of zero at the outlet as shown in Figure 2. The pipe was conservatively considered to be plugged at the upstream end. The stiffness of the pipe and shearing resistance of the adjacent and overlying soil was neglected; thus, the only resistance to uplift was provided by the weight of the pipe and the overlying soil. The soil unit weight was assumed to be 18.85 kN/m^3, roadway width 10 m shoulder to shoulder, and the embankment slope extending from the bottom of the culvert to the edge of the shoulder at a slope of 1 vertical to 2 horizontal. Cover above the pipe was taken as 610 mm; the pipe is not beveled. With these constraints, the pipe length and embankment height increases with increasing pipe diameters.

The forces used in the analyses are shown in Figure 2 where W_s is the weight of the soil cover, W_p is the weight of pipe, U is the resultant of the pore pressure distribution, and R is the resistance of the tie-down. The soil and pipe weight resultants act through the center of the embankment and the pore water force acts through the centroid of the triangular pore water pressure distribution, which is at a distance of one-third the pipe length measured from the inlet. The required resisting force is calculated from:

$$R = U - W_p - W_s$$

Obviously, this is a very simplified analysis that provides resisting forces that are conservatively high, because it ignores the CMP stiffness and the soil-structure interaction.

Force Comparison of Various Restraints

For each agency's standard, the resisting force of the restraint was computed for a range of pipe diameters and with a soil cover depth of 610 mm. These relationships between the resisting forces and pipe diameters can be classified as either linear or exponential shaped curves and are shown in Figures 3 and 4. In all cases but one, the last point on the curve represents the maximum diameter CMP that the standards allow.

Figure 3 shows resisting force versus pipe diameter for the standards in which the relationship between pipe diameter and resisting force is linear. Also shown is the relationship resulting from the simplistic, conservative analysis (*3*). All of the standards with a linear relationship between force and diameter have much lower forces than those calculated by the rational analysis. The agency with the

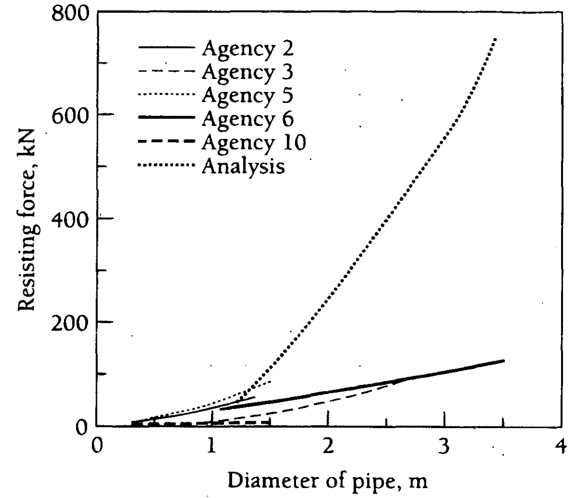

FIGURE 3 Linear relationship between resisting force and pipe diameter from various DOT specifications.

lowest forces in its standards is also the only one which had an uplift failure when restraint was used.

Agency standards that have an exponential relationship between resisting forces and pipe diameters are shown in Figure 4. Although the curve from the simplistic analysis very is conservative, only one standard in this group has lower forces. The other three standards have resisting forces that are equal to or exceed those of the analysis.

This synthesis points out the diversity in resisting forces among these agencies with a 500 percent variation for pipes about 2 m in diameter. It appears some standards may be providing resistance to uplift that is dangerously low whereas others are extremely conservative and may be too restrictive. The force comparison reinforces the interpretation that existing standards are not based on experimental results nor on rigorous theoretical analyses.

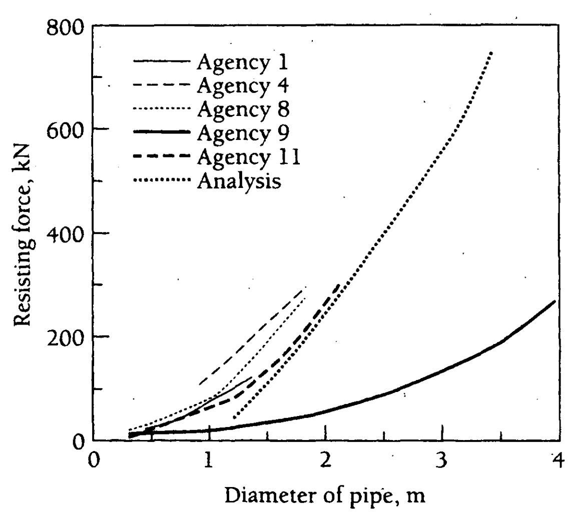

FIGURE 4 Exponential relationship between resisting force and pipe diameter from various DOT specifications for tie-downs.

CASE HISTORIES

As part of the survey of Iowa counties, a few uplift failure sites were identified in which additional data were available for analysis. Two such sites are described here. Unfortunately data on prior rainfall, flooding, and maintenance were not available.

Site 1

This site had a corrugated structural steel plate culvert installed in February 1954 that failed in June 1976. The pipe was circular with a diameter of 3.66 m, a length of 29 m, a beveled inlet 1.5 to 1, and a slope of 3.81 percent. The fill above the pipe was 853 mm deep and the roadway 8.53 m wide with a slope of 8 to 1 at the inlet and 6 to 1 at the outlet. The fill extended to the top of the beveled inlet. No information was found on high water marks or discharge through the pipe during the flow that caused the failure.

The pipe bent at a distance of 7.92 m from the inlet until it was at about the same elevation as the shoulder of the roadway. This type of bending failure is illustrated in the photograph of Figure 5*a*. The pipe bottom collapsed inward beginning at about 6.71 m upstream of the centerline of the road and extended approximately to the centerline. Subsequent to the bending, the road grade washed out, but it was not clear whether water overtopped the road or undermined the pipe to cause the embankment failure. No tie-down structure or cutoff wall was used.

Shear and moment diagrams were constructed from the soil loading. The maximum shear was found to occur at a distance of 8.84 m from the inlet. The bend in the pipe at a distance of 7.92 m occurred close to the point of maximum shear. The moment that must be resisted at 8.84 m is 643 kN.m and could be resisted by a force of 75.2 kN located 305 mm from the inlet. In this case, the calculated resisting force for this 3.66-m diameter pipe falls considerably below the extrapolated resisting forces specified by any of the state or provincial agencies.

Site 2

The CMP at Site 2 was installed in July 1976 and failed in September 1986. The pipe was a structural plate pipe with a diameter of 3.05 m, a length of 36.6 m, a slope of 3.67 percent, and a projecting inlet. The roadway width was 8.53 m with slopes of 2.5 to 1 on both inlet and outlet ends and a depth of soil cover averaging 850 mm. A seepage collar was place 6.1 m downstream from the inlet. A tie-down consisting of two wood piles driven on each side of the pipe with two 76 × 406-mm horizontal wood planks across the top of the CMP and a 13-mm wire rope cable attached to the piles. Figure 5*b* shows a similar tie-down that failed by pulling the piles out of the ground.

At Site 2 no data on high water or discharge were available, but the pipe failed by breaking the planks of the tie-down and stretching the cable. The broken tie-down at Site 2 is shown in Figure 5*c*. The road grade washed out and the culvert was moved 100 m down stream. The bottom of the pipe collapsed inward so there was only about 610 mm of clearance between the top and bottom of the failed pipe.

The force required to break the tie-down was estimated from available data. The flexural strength of the wood plank was assumed to be 49.6 MPa and the moment of inertia calculated to be 57.1 m^4.

a. FAILED CMP WITHOUT TIE-DOWN SYSTEM.

b. PULL-OUT OF TIE-DOWN SYSTEM.

c. FAILED TIE-DOWN SYSTEM.

FIGURE 5 Examples of CMP uplift failures.

Using the flexural equation, the failure moment is estimated to be 160 kN.m. This moment corresponds to a uniform load of 52.5 kN acting over 1.52 m, the length where the pipe and planks are in contact. The indentation in the pipe made by the planks was obvious, so the total force acting on the planks is estimated to be 262 kN. In addition to the strength of the planks, the stretch of the cable must be included. If the cable is 13-mm wire rope with a yield stress of 1724 MPa, the ultimate load would be 160 kN; however the cable was not new so its ultimate strength was reduced by 20 percent to account for corrosion. Thus, the two cables could carry an addition load of 258 kN making the total load to fail the tie-down estimated to be 520 kN. This failure load for the 3.05 m diameter pipe is greater than the specified resisting forces for 6 of the 9 specifications studied and approaches the force calculated in the simplified, conservative analysis.

The history of this pipe also points to the limitations of seepage cutoffs in eliminating uplift on CMP. It is often thought that cutoff walls and graded bedding are sufficient to mitigate CMP uplift problems; however, theoretical flow net analyses indicate that a cutoff needs to extend approximately 80 percent of the distance to an impermeable layer in order to reduce the quantity of seepage by 50 percent (*4,5*) and that incorrect placement of cutoffs can result in a redistribution of pore pressures that might exacerbate the problem (*5,6*). From a practical standpoint differential settlement, cracking, perforation, and burrowing animals can ruin the best designed seepage control system.

CONCLUSIONS

Between 1970 and 1975 and between 1983 and 1988, the secondary road system in Iowa reported approximately six CMP uplift failures per year. Uplift failures of CMP throughout North America are fairly rare with only 17 percent of U.S. states and Canadian provinces reporting nine failures between 1987 and 1992. Failures were limited to pipes with diameters greater than 1.52 m. Three DOTs do not use CMP, and six specify a maximum diameter of CMP of 1.37 to 2.13 m. Although the number of reported failures may appear low, this record is unacceptable when there is potential for loss of life and when it is possible to alleviate the problem with proper end treatment. Of those reporting failures, only one state had used end restraint standards. Twenty-six of 52 agencies have standards for end protection.

Although FHWA and some state DOTs have specifications for flexible pipe tie-downs or headwalls, the bases for their recommendations are not clearly defined. Further, few field data on pore pressure distributions, or uplift loading conditions are available; however, through the analysis of two CMP failures where original designs and post failure measurements were available, the premise that uplift was caused by pore pressures appears reasonable. In one case history, where no end restraint was used, failure occurred at a force less than the least conservative specification. In a second case history, in which end restraint was used, the force to cause failure approached the more conservative specifications.

Of those agencies that provided data to compare end restraint force as a function of CMP diameter, five have lower resisting forces than those computed by a simplistic analysis and three have forces approximately equal or slightly greater. The large range in these standards and the continuation of uplift failures suggest that experimental work including pipe longitudinal stiffness and soil-pipe interaction of pipes greater than 2 m in diameter is needed to develop a rational set of specifications for end restraint. Two studies addressing these issues are reported as part of these proceedings (*7,8*).

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REFERENCES

1. Edgerton, R. C. Culvert Inlet Failures—A Case History. *HRB 286, 1960.*
2. *Pipe Culvert Inlet and Outlet Protection.* FHWA Notice N 5040.3, FHWA, U.S Department of Transportation, April 1974.
3. Austin, T. A., R. A. Lohnes, F. W. Klaiber. *Investigation of Uplift Failures in Flexible Pipe Culverts: Final Report.* ERI Project 3024, ISU-ERI-Ames-90227, Ames: Engineering Research Institute, Iowa State University, 1990.
4. Goehring, R. L. *Preliminary Structural and Hydraulic Analysis of Flume Bridges in Western Iowa.* M.S. thesis. Iowa State University, Ames, Iowa, 1981.
5. Cedegren, H. R. *Seepage, Drainage and Flow Nets*, 2nd ed. John Wiley & Sons, Inc., New York, 1977, 535 pp.
6. Lambe, T. W., and R. V. Whitman, *Soil Mechanics, SI Version.* John Wiley & Sons, Inc., New York, 1979, 553 pp.
7. Haven, B. T., F. W. Klaiber, R. A. Lohnes, and L.W. Zachary. *Longitudinal Strength and Stiffness of Corrugated Metal Pipe.* Presented at 73rd Annual Meeting of the Transportation Research Board, Washington, D.C., 1994.
8. Kjartanson, B. K., R. A. Lohnes, F. W. Klaiber, and B. T. McCurnin. *A Full-Scale Field Test of the Uplift Resistance of a Corrugated Metal Pipe Culvert.* Presented at 73rd Annual Meeting of the Transportation Research Board, Washington, D.C., 1994.

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