T. Al Austin Robert A. Lohnes F. Wayne Klaiber

Investigation of Uplift Failures in Flexible Pipe Culverts

Sponsored by the Iowa Department of Transportation
Highway Research Advisory Board

March 1990

Iowa DOT Project HR-306
ISU-ERI-Ames-90227

Iowa Department of Transportation

report

College of Engineering
Iowa State University

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ABSTRACT

This study was precipitated by several failures of flexible pipe culverts due to apparent inlet floatation. A survey of Iowa County Engineers revealed 31 culvert failures on pipes greater than 72" diameter in eight Iowa counties within the past five years. No special hydrologic, topography, and geotechnical environments appeared to be more susceptible to failure. However, most failures seemed to be on pipes flowing in inlet control. Geographically, most of the failures were in the southern and western sections of Iowa. The forces acting on a culvert pipe are quantified. A worst case scenario, where the pipe is completely plugged, is evaluated to determine the magnitude of forces that must be resisted by a tie down or headwall. Concrete headwalls or slope collars are recommended for most pipes over 4 feet in diameter.

ACKNOWLEDGEMENTS

This study was sponsored by the Highway Research Advisory Board, Iowa Department of Transportation under Research Project HR-306. The authors extend sincere appreciation to Darrell Coy, Iowa DOT for his assistance in the conduct of this project. A special thanks go to the Iowa county engineers who provided valuable information concerning flexible pipe culverts in their counties.

Special thanks are accorded to graduate students Michael Chyi, Dipak Patel, and Steve Van Welding for their contributions to this project.

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1. PROBLEM DEFINITION

Flexible pipe culverts (corrugated metal pipes) are important components of the drainage systems associated with Iowa's road system. Many county engineers have used large diameter flexible pipe culverts to replace small bridges and have realized significant savings to the counties. However, there is a perception that in some situations, these flexible pipe culverts have not performed adequately. In late 1987, the authors met with several Iowa Department of Transportation (Iowa DOT) personnel to discuss research needs regarding apparent uplift failures of large corrugated metal pipe (CMP) culverts. This meeting led to submittal of a problem statement, and subsequently a research proposal to the Highway Research Board, Iowa DOT. See Appendix A for photographs of several recent culvert failures. This report summarizes the findings of this research program and makes recommendations for immediate action and future research.

1.1. Iowa DOT Survey (1975)

A survey of Iowa county engineers, conducted by the Iowa DOT in 1975, attempted to determine the extent of CMP culvert failures in Iowa. About 50% of the county engineers in Iowa responded. No additional follow-up to the questionnaire was made. Table 1.1 shows the results of the survey.

Table 1.1. Survey Results Iowa DOT, 1975

Pipe Size (Inch)	Number of Structures Projecting Inlets	Beveled Inlets	Number of Failures Projecting Inlets	Beveled Inlets
60 to 96	226	166	2	11
97 to 120	19	46	1	5
121 up	11	53	2	9

While this survey included only 50% of the counties, the results were surprising in that the percentage of the pipes that failed were higher than expected. In pipes less than 96"in diameter, five percent or less of the reported culverts had failed, but in the larger diameter pipes the percentages of failures were significantly higher. The questionnaire asked for experiences with CMP culverts that were installed within the past five years and did not differentiate between floatation (uplift) failures and fold over failures in beveled or step beveled inlets. The 1975 Iowa DOT survey resulted in the issuance of a letter to all county engineers, dated February 20, 1976, from C. Pestoknik in which he states:

> "...we feel that if you get a design highwater at the inlet of an unprotected structure, the chances are too high that it will float or fold over. Therefore we are suggesting that you anchor and reinforce the inlet ends of unprotected structures as soon as possible." (Pestoknik, 1976)

At about the same time the problem of flexible pipe culvert failures were being recognized as a problem in Iowa, the Federal Highway Administration issued a FHWA Notice N 5040.3, dated April 26, 1974, that addressed the problem of pipe culvert inlet and outlet protection. This notice stated:

> "Positive engineering attention should be given to the need for providing protection at the ends of all pipe culverts having a height of 48 inches and larger." (FHWA, 1974)

Enclosed with the FHWA Notice were headwall and slope paving design standards for circular and slope tapered culverts up to 180 inches in diameter. The Iowa DOT included the FHWA headwall and slope paving standard designs with the 1975 Survey results sent to all Iowa County Engineers and suggested the county engineers adopt some type of tie down structures, such as, the FHWA standards.

Despite the above efforts by the Iowa DOT and the FHWA, reports of CMP culvert failures continued to arrive at the Iowa DOT headquarters. Concern that current design and/or construction practices were not adequate led to the development of this project.

2. OBJECTIVES

The ultimate goal of this project is to eliminate or significantly reduce uplift failures in CMP culverts through improved design of new structures and retrofitting of existing culverts. Elimination or reduction of uplift failures can be realized only if certain intermediate objectives are met. The objectives of this project are:

- define the hydrologic, topographic, and geotechnical environments and pipe geometries most conducive to CMP culvert uplift failures,
- identify the mechanism(s) that causes uplift and subsequent failure of CMP culverts, and
- determine the magnitude and distribution of the forces that are likely to cause flotation of CMP culverts.

The research plan included a new survey of Iowa county engineers to obtain more specific information about the number of

CMP failures and the hydraulic, geotechnical and structural environments associated with each failure. Data on tie downs, anchors and cutoffs that are being used were also to be collected. These data were to be used to develop and evaluate hypotheses about a "worst possible case scenario" and "a most likely to be stable scenario". Based on these scenarios, evaluations of the potential loading on the culvert was developed. The amount of resisting force located at the inlet of the pipe necessary to maintain structural equilibrium was determined for a range of geometric conditions. "Post mortem" evaluations of two failures for which sufficient data were available were conducted to quantify the loadings derived.

3. 1988 SURVEY RESULTS

A survey questionnaire was sent to all Iowa County Engineers in April 1988 requesting information on the number of culverts that had failed due to inlet floatation or fold over. Sixty eight questionnaires were returned completed (69% of the counties). Eight counties (12 % of those reporting) indicated they had one or more culvert failures during the past five years. This compares to eight counties reporting failures (16% of the 50 counties reporting) in the 1975 Iowa DOT Survey. Despite the use of various tie down structures, CMP failures are still occurring. Of the counties reporting failures, 75% indicated that they used some form of tie down, including pile and cable tie downs, tied concrete curtain walls, concrete slope collars, and tied sheet piling cut-off wall structures. Therefore, it appears that many forms of tie downs being utilized have not solved the uplift problems and thus improved designs are still needed.

A total of 31 culvert failures were reported on the survey. Table 3.1 shows the ranges of culvert sizes shown on the survey forms. The total number of culverts shown in Table 3.1 is less than 31 because some sizes had more than one failure. Failures have occurred with both circular pipes and elliptical pipe arches and with both projecting inlets and beveled or step beveled entrances. The majority of respondents indicated they think plugging or partially blocking of the inlets contributed to the uplift failure.

Table 3.1. Culvert Failures by Size and Entrance Condition

Diameter (in)	Length (ft)	Entrance Condition
72	88	Unknown
78	108	Beveled
90	120	Projecting
102	54	Beveled
102	62	Unknown
108	70	Beveled
108	125	Projecting
128" x 83"*	146	Beveled
132	120	Projecting
138	72	Beveled
144	90	Projecting
14'10" x 9'7"*	152	Beveled
14'10" x 9'7"*	UNK	Beveled
180	120	Unknown
204	96	Unknown
32.1' x 19.2*'	260	Unknown

* Elliptical pipe arch

No unique geologic or hydrologic conditions could be identified that characterized the majority of the failure sites. The problem appears to be more common in those regions of the state where significant elevation drop exists across the culvert, and the downstream river valley would yield low tailwater. Also, the problems seem more common in areas of the state where loess derived soils occur.

Field trips were made to seven county that responded with failures to the survey and data on failures were obtained. Two sites will be discussed in detail later in the section on Case History.

A brief collection of photos from Iowa DOT staff and those taken by the principal investigators on this project are included in Appendix A.

4. HYDRAULIC CONSIDERATIONS

An analysis of the hydraulics of flow into and through culverts is necessary to understand the various loadings that may lead to uplift failures. A culvert represents a reduction in cross sectional area of flow for the approaching water; therefore, the velocity of the water in the culvert must be increased proportionally to the reduction in cross sectional area. In order to gain the energy needed to accelerate the flow, an increase in potential energy upstream of the culvert must occur. This increase in potential energy is developed by a rise in the water level upstream of the culvert. This headwater also provides the pressure for uplift of the pipe inlet, if the water is able to saturate the material under the pipe. The following sections will discuss the parameters that affect the headwater at any culvert inlet.

4.1. Types of Flow

Flow through culverts is a complex hydraulic problem; however, in general flow can be classified in several simplified ways. The most widely used classification is based on the location of the hydraulic control section; that is, inlet control where the hydraulic control section is at the culvert inlet and outlet control where the hydraulic control is located at the culvert outlet. Inlet control exists when the culvert barrel has a greater capacity to transmit flow than the inlet will accept. Outlet control occurs whenever the culvert barrel cannot transmit as much flow as the inlet opening will accept. At low flows, culverts generally function in inlet control; however, during a storm as the flow rate and headwater elevation increases, a culvert may shift from inlet control to outlet control. In culvert design, the engineer is interested primarily in the flow control and headwater elevation at the design flow rate.

In culverts flowing in inlet control, critical depth, the depth at which specific energy is minimized, will occur near the entrance of the culvert and flow in the culvert barrel will be shallow, high velocity flow (super critical) through all or some part of the barrel. Under inlet control the downstream hydraulic conditions will not affect the culvert capacity. Most often inlet control exists for culverts with relatively steep slopes and/or low tailwater conditions.

In outlet control the headwater is dependent on the tailwater conditions, the friction loss in the pipe and the entrance condition. Downstream conditions affect the headwater upstream of the culvert. Culverts flowing in outlet control generally have flatter slopes than those in inlet control, higher tailwater depths and frequently the culvert barrel is flowing full or near full.

The weight of the water within the culvert serves as a resisting force against the uplift pressures, so culverts that are flowing full or near full have the largest resisting force against uplift. The problems of inlet uplift appear to be associated with culverts flowing in inlet control since in inlet control the flow is supercritical through all or most of the culvert barrel resulting in depths of flow in the culvert barrel that are less than the headwater depth and most often are less than the diameter of the pipe. Uplift effects will be increase if the inlet is blocked or partially blocked with debris because the depth of flow, and therefore the weight of the water, in the culvert will be reduced.

4.2. Entrance Conditions

A large number of possible entrance configurations are available for culvert inlets. Commonly used culvert entrances included projecting inlets where the culvert extends from the fill; concrete headwalls either with or without wingwalls to assist the flow transition; beveled, step beveled or mitered to conform to the slope of the fill; and prefabricated or precast end sections (See Figure 4.1). Each entrance condition has different hydraulic

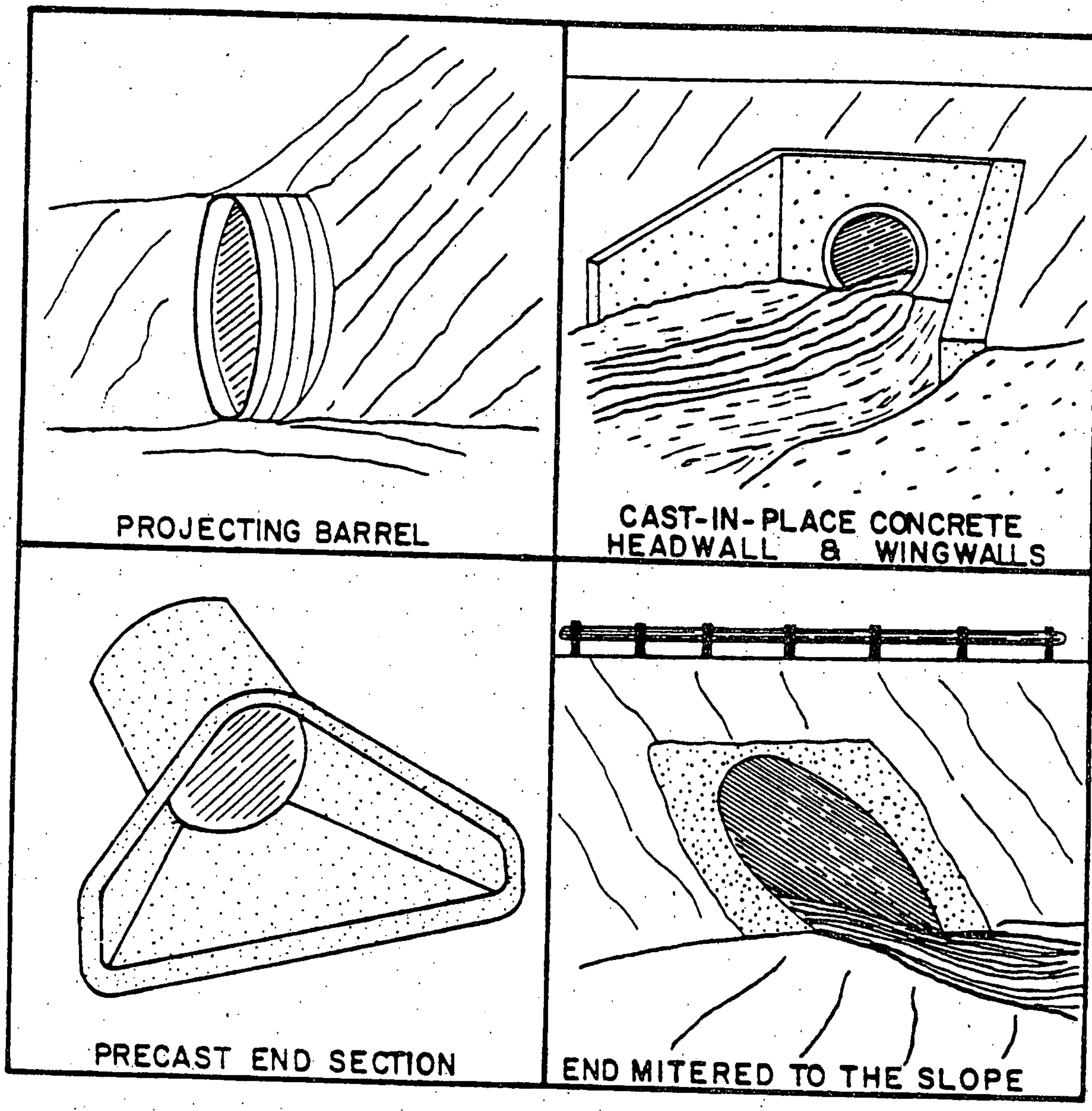


Fig. 4.1 Typical inlets for culverts (HEC 5, 1985)

properties that can be estimated using techniques such as those outlined in "Hydraulic Design of Highway Culverts" (HEC-5, 1985).

In Iowa, most CMP culverts are either projecting inlets, step beveled inlets or use a standard CMP end section. In general, at a constant flow rate through the culvert, the headwater depths will be greatest for projecting inlets followed by step beveled inlet and standard end section, in the order of decreasing headwater. Projecting inlets, especially for large diameter pipes, project a significant pipe distance uncovered by the fill. For example, a 12 feet diameter circular pipe projecting from a fill with 3 horizontal to 1 vertical side slopes will extend out of the fill 36 feet. The force available to resist uplift pressures in this uncovered length consists only of the weight of the pipe and water within the pipe. This condition creates the most severe situation possible for uplift failure.

Pipes with step beveled inlets generally have the fill extending to the top of the pipe with little or no uncovered pipe. The step bevel that Iowa uses consists of a vertical cut 1/4 of the diameter on the top and bottom of the pipe and a sloping section between (See Figure 4.2). The step bevel inlet improves the hydraulic efficiency of the inlet and decreases the flow contraction that occurs in the inlet. An added advantage of step beveled inlets is the increased resisting force due to the extra weight of the fill on top of the inlet since it is not projecting from the fill. However, cutting the pipe in a step bevel reduces the internal resistance of the pipe to deformation.

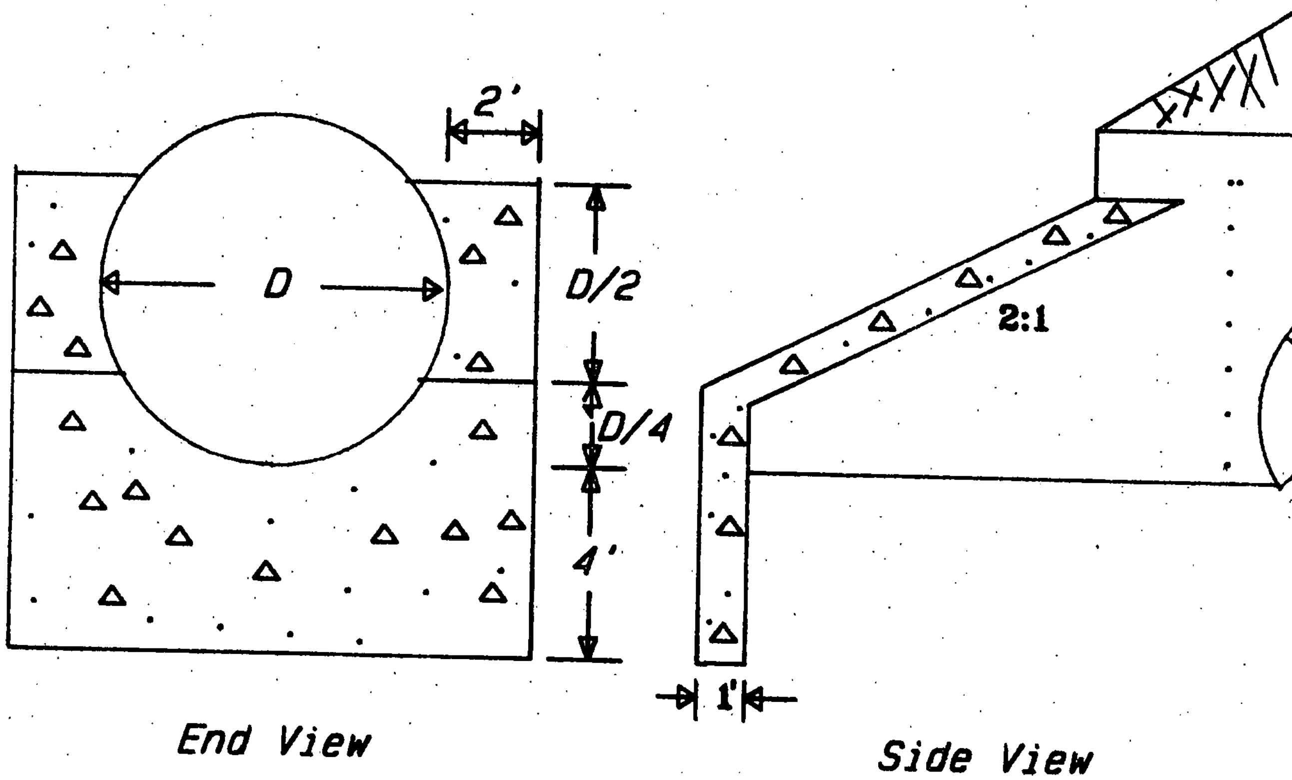


Fig 4.2 Schematic of headwall currently remmended by Iowa DOT for step beveled inlets

Headwalls at culvert inlets allow not only the fill to come to the top of the pipe, but also provides an extra concrete mass at the pipe entrance to resist the uplift pressure. The same affect can be obtained through concrete slope paving around step beveled inlets. Adequate provision for attaching the pipe to the headwall must be provided if the pipe and headwall are to resist uplift together. In some cases, partial headwalls, extending only 1/3 to 1/2 of the pipe diameter have been used to provide extra concrete mass to resist uplift.

4.3. <u>Computer Program</u>

A computer program was developed as part of this research to estimate the water surface profile through a culvert with various entrance conditions and flow controls. This computer model uses the gradually varied open channel flow equation and the direct step method to estimate the water surface profile through a culvert. It is recognized that at some locations in a culvert the flow condition may be rapidly varied, especially near the entrance to the culvert. Rapidly varied flow is not included in the computer model.

A typical gradually varied flow situation in an elemental length of a culvert is shown in Figure 4.3. This situation assumes the culvert is not flowing full as will be the case in most inlet control situations and in some outlet control situations. Applying the energy equation at section 1 and section 2 in Figure 4.3 (in the direction of flow) gives:

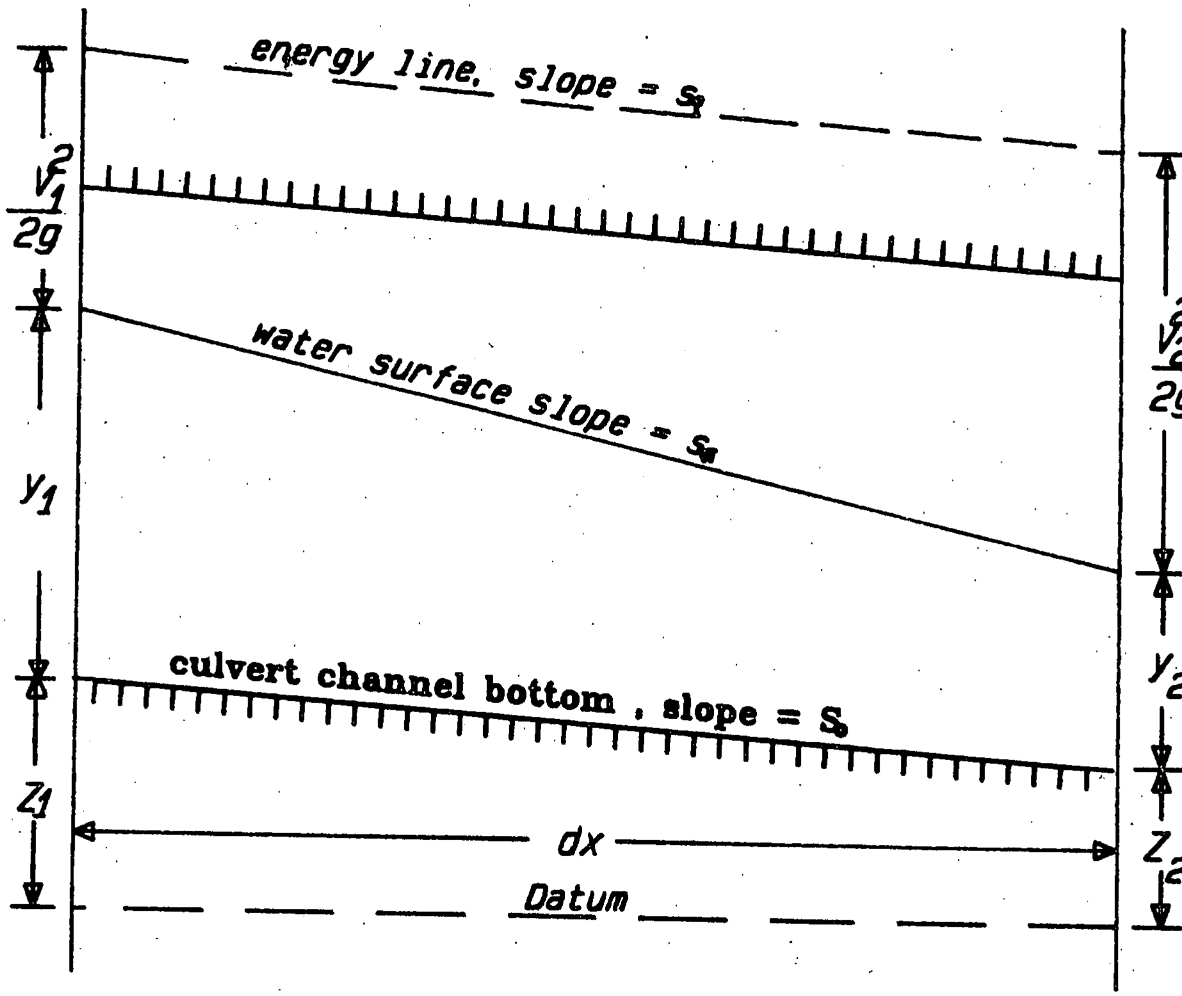


Fig. 4.3 Derivation of the gradually
varied flow equation

$$(V_1^2/2g) + y_1 + z_1 = (V_2^2/2g) + y_2 + z_2 + h_f \qquad 4.1$$

where:

V_1 and V_2 are velocities at sections 1 and 2 respectively,

y_1 and y_2 are depth at section 1 and 2 respectively,

z_1 and z_2 are invert elevations at section 1 and 2 respectively, and

h_f is the headloss between sections 1 and 2.

The slope on the energy gradeline, s_f, is headloss per unit length of pipe or h_f/dx. Thus $h_f = s_f\,dx$. The slope of the channel bottom, s_0, is the difference in elevation per unit pipe length or $(z_1 - z_2)/dx$. Thus, $(z_1 - z_2) = s_0\,dx$. Rearranging equation 4.1 and substituting for h_f and $z_1 - z_2$ gives:

$$s_0dx + (y_1 - y_2) + (V_1^2 - V_2^2)/2g = s_fdx \qquad 4.2$$

The specific energy, E, in open channel flow is the depth of flow, y, plus the velocity head $(V^2/2g)$. Thus Equation 4.2 can be rewritten as:

$$dx = (E_1 - E_2)/(s_o - s_f) \qquad 4.3$$

Equation 4.3 can be solved using the direct step method. In the direct step method, the computations begin at a hydraulic control where the depth and velocity of the flow is known and proceed either upstream for subcritical flow (depths of flow greater than critical depth) or downstream for supercritical flow (depths of flow less than critical depth). If the design discharge, culvert size and material, and culvert entrance condition have been determined, all hydraulic properties at the control section can be found. Assume, the control section becomes section 1 in equation 4.3. The specific energy, E_1 can be determined since the depth and velocity must be known at the control section. A depth, y_2, is assumed at some other section; however, the location of this section is not known at this time. The velocity can be found from the assumed y_2, the culvert shape, and the design discharge using the continuity equation. The distance upstream or downstream to the point where the depth of flow is y_2 is then calculated using equation 4.3. The slope on the energy gradeline, s_f, is determined using a uniform flow equation, such as the Manning Equation.

$$S_f = [\, V\, n\, /(1.486\, R^{2/3}\,)\,]^2 \qquad 4.4$$

where:

V = velocity
n = Manning roughness dependent on culvert material
R = Hydraulic radius = A/P
A = area perpendicular to flow
P = Wetted perimeter

Once the distance upstream, dx, is calculated, the section 2 in equation 4.3 becomes a "new" section 1 for the next step computation. Repeated application of equation 4.3 in this manner provides computation of the entire surface water profile through the culvert. Once the profile is determined, the weight of water in the pipe can be found by simple geometry.

4.4. Classification of Flow Profiles

For a given discharge and culvert, the normal depth and critical depth can be determined. Normal depth is the depth corresponding to uniform flow or the depth where the energy lost due to friction equals the energy gained through a change in elevation. Critical depth is the depth that corresponds to the minimum specific energy at a given flow rate. For hydraulically mild sloping channels normal depth is greater than critical depth. For hydraulically steep sloping channels normal depth is less than critical depth. If the slope on a channel is equal to the critical slope, normal depth and critical depth are equal. Thus, critical and normal depths are used to divide the possible flow profiles into three zones based on the relationship between the depth of flow in the channel and critical and normal depth (Figure 4.4). If the depth of flow is above the upper line the flow profile is in Zone 1 (i.e. M-1 or S-1 on Figure 4.4). If the depth is between normal and critical depth the profile is in Zone 2 (i.e. M-2 or S-2 on Figure 4.4), and if the depth of flow is below the lower line, the flow profile is in zone 3 (i.e. M-3 or S-3 on Figure 4.4). Figure 4.4 shows the flow profiles used in the model developed in this project. It is necessary to determine the flow profile applicable to each situation so the appropriate hydraulic control section can be located.

A culvert will flow full when the outlet is submerged or when the outlet is not submerged but the headwater is high and the barrel is long. According to laboratory investigations, the entrance of an ordinary culvert will not be submerged if the

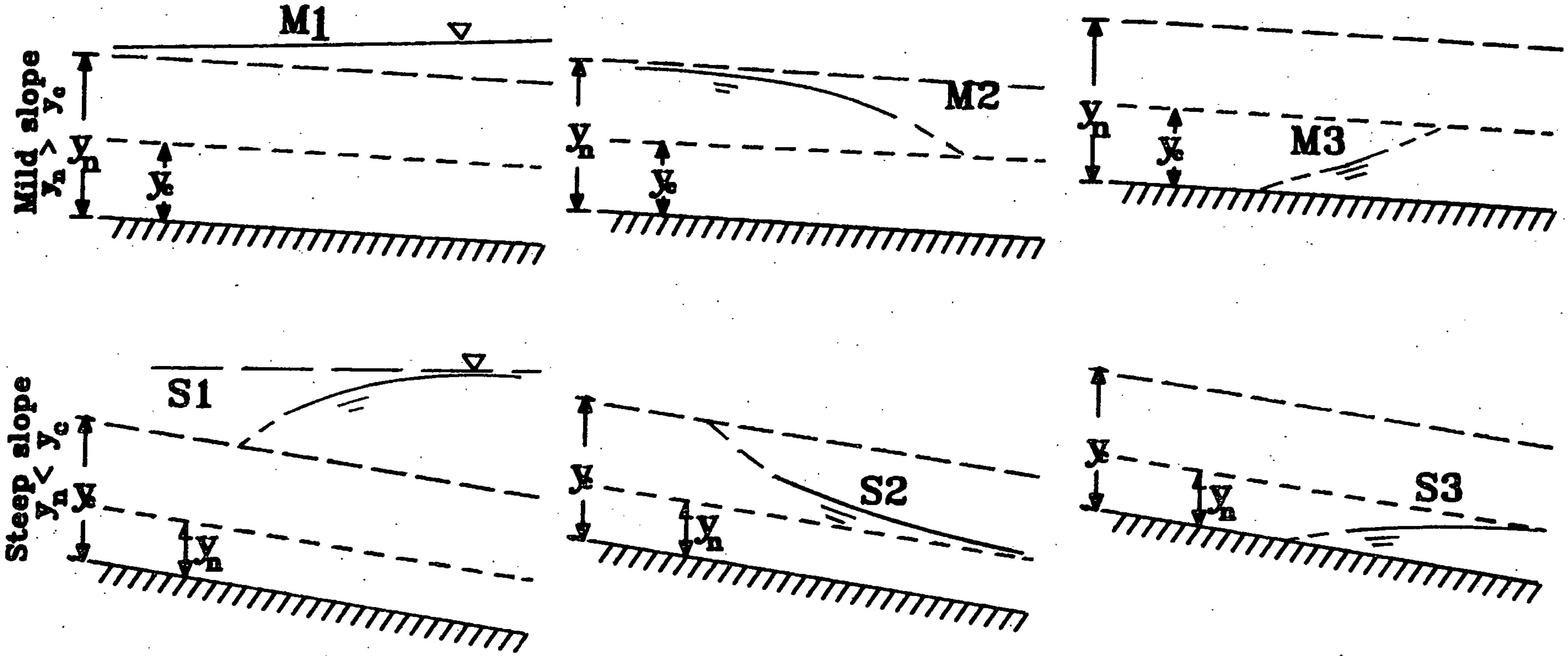


Fig. 4.4 Water Surface Profile

headwater/diameter ratio (HW/D) is less than 1.2 to 1.5. For this project it was assumed the culvert was flowing full whenever the HW/D ratio is 1.5 or greater.

For this research, culvert flow is divided into eight types as shown in Figure 4.5. Table 4.1 shows the pipe slopes, tailwater depths, headwater /diameter ratio and water surface profile type associated with the eight flow profiles shown in Figure 4.5.

4.5. Simulation of Water Surface Profiles

Three computer programs were written as part of this research. All models use Turbo Pascal computer language and will operate on most personal computers. CULVERT simulates the water surface profile for Types 2 to 7 (Table 4.1) for circular pipe culverts. This program is also capable of plotting the water surface profile on the screen, printer or plotter. CULVERT calculates the critical

Table 4.1. Flow Profile Classifications for Culvert Flow used in this Project

Profile Type	Pipe Slope[1]	Tailwater Depth[2]	Headwater/depth	Water surface Profile
1	M or S	>D	>1.5	Undefined[3]
2	M	$<d_c$	>1.5	M-2
3	S	$<d_c$	>1.5	S-2
4	S	$>d_c$	<1.5	S-1
5	M	$>d_n$	<1.5	M-2
6	S	$<d_c$	<1.5	S-2
7	M	$>d_n$	<1.5	M-1
8	M or S	<D	>1.5	Undefined[3]

[1]M= mild slope $d_n > d_c$; S= steep slope $d_c > d_n$
[2]D= pipe diameter or height; d_c = critical depth; d_n = normal depth
[3]Undefined means pipe is flowing full and no water surface profile exists.

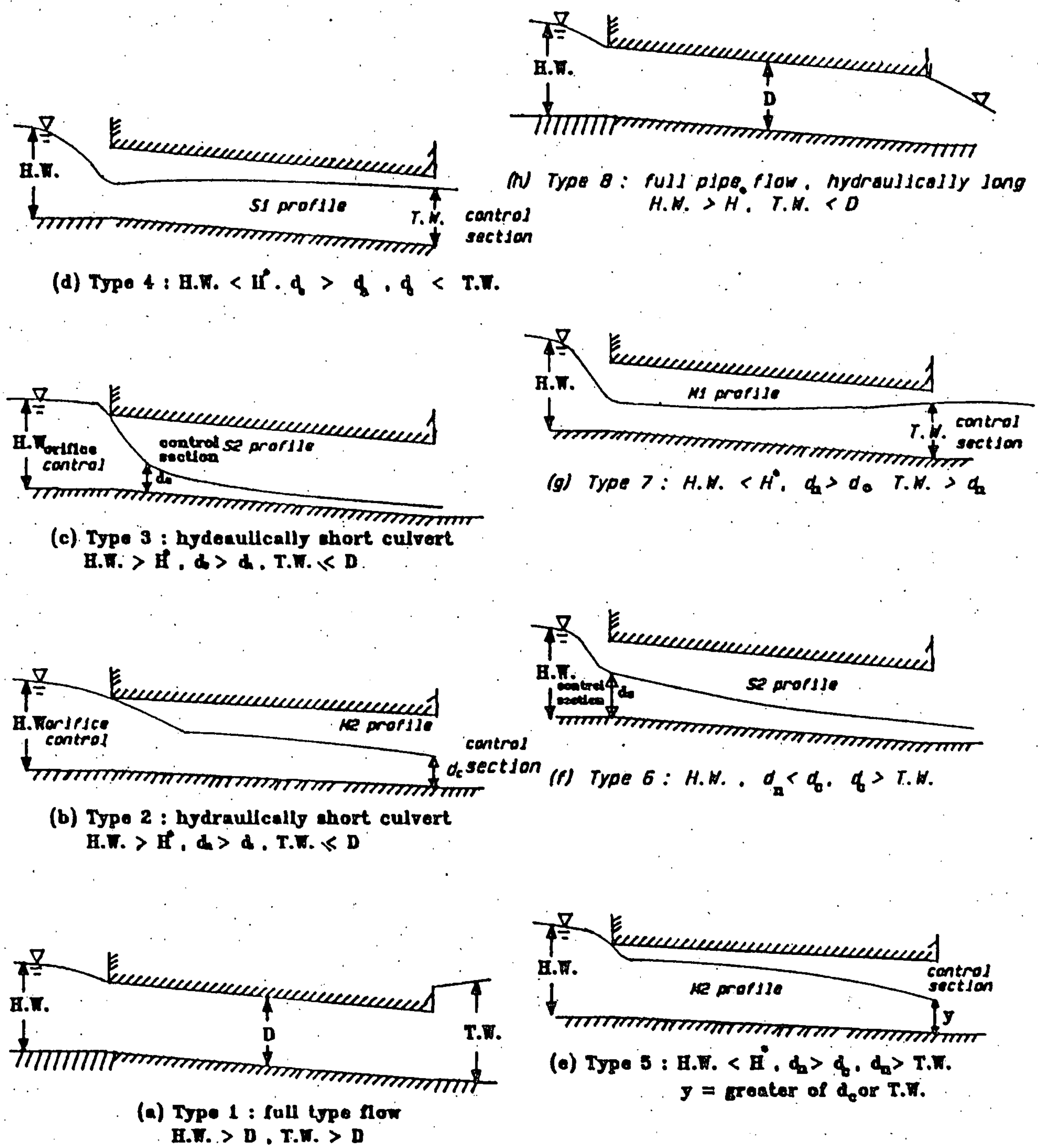


Fig. 4.5 Water Surface Profile included in Computer Model

embankment depth that would be required to resist the uplift pressures if the pipe were acting as a simple beam.

CULVERT.2 also simulates the water surface profile for the same conditions as CULVERT, but in addition, CULVERT.2 calculates the moment about the inlet of the culvert at one foot intervals along the culvert length. CULVERT.2 is capable of plotting the water surface profile, and the moment versus pipe length diagram on the screen, printer or plotter.

CUL-FLOW is used to determine the flow rate coinciding with a given headwater depth, pipe slope, pipe diameter, tailwater, Manning's "n", and entrance conditions. This model evaluated both inlet and outlet control to determine the minimum flow rate that results in the given headwater depth.

Listings of the source codes for these three programs are included in Appendix B.

4.6. Use of the Models

The models have been tested on a culvert site where a failure occurred due to uplift. The culvert was a 12 feet diameter CMP on a slope of 3.8%. The models were used to determine the water surface profile for various assumed headwater and tailwater depths. Once the profile was computed, the CULVERT.2 model was used to calculate the moments diagram for the pipe. Since the roadway cross-section was fixed, the only additional resisting force against uplift was due to the weight of the water in the pipe. Ignoring any pipe strength, the pipe was determined to be either "safe" when the resisting force exceeded the uplift force or "fail" when the resisting force was less than the uplift force. Table 4.2

Table 4.2. Example of Culvert Hydraulic and Stability Analysis

Diameter (ft)	Discharge (cfs)	Headwater (ft)	Tailwater (ft)	Stability Condition
12	270	5.0	1.0	Safe
12	270	5.0	2.0	Safe
12	270	5.0	3.0	Safe
12	270	5.0	4.0	Safe
12	340	5.7	1.0	Safe
12	340	5.7	2.0	Safe
12	340	5.7	3.0	Safe
12	340	5.7	4.0	Fail
12	440	7.0	1.0	Safe
12	440	7.0	2.0	Fail
12	500	8.0	1.0	Fail
12	500	8.0	2.0	Fail
12	670	9.0	1.0	Fail
12	670	9.0	2.0	Fail
12	785	10.0	1.0	Fail
12	785	10.0	2.0	Fail
12	870	11.0	1.0	Fail
12	870	11.0	2.0	Fail
12	1040	12.0	1.0	Fail
12	1160	13.0	1.0	Fail

shows the results of these simulations. The analyses presented in Table 4.2 are very conservative (See the Pore Water Analysis Section of this report) and are presented here for demonstration only.

The analyses of the above culvert site determined the culvert to be unstable (Fail) at headwater depths greater than seven feet (HW/D > 0.58). No tie down was included at this culvert site and the culvert was assumed to have a projecting inlet. For headwater depths lower than seven feet, the stability was a function of the tailwater depth.

5. PORE PRESSURE ANALYSIS

5.1. Objective and Assumptions

The objective of the uplift pressure study is to obtain an estimate of the magnitude of uplift forces and the location of the resultant. The assumptions for these analyses are:

1) the pipe is treated as a beam but no consideration is given to its longitudinal flexural strength (stiffness), the only resistance from the pipe is its weight;

2) soil shear strength or deformation characteristics are ignored, only the soil weight is considered and the unit weight is assumed to be 120 lb/ft^3.;

3) Variations in the flow line of the water in the conduit are not calculated, in most cases the analysis assumes a plugged pipe with no flow in some analyses the pipe is assumed to be flowing 50% or 75% full with a uniform flow line;

4) The headwater elevation is at the top of the pipe on the upstream end and the tailwater is at the bottom of the pipe;

5) the pore pressure is dissipated linearly beneath the pipe;

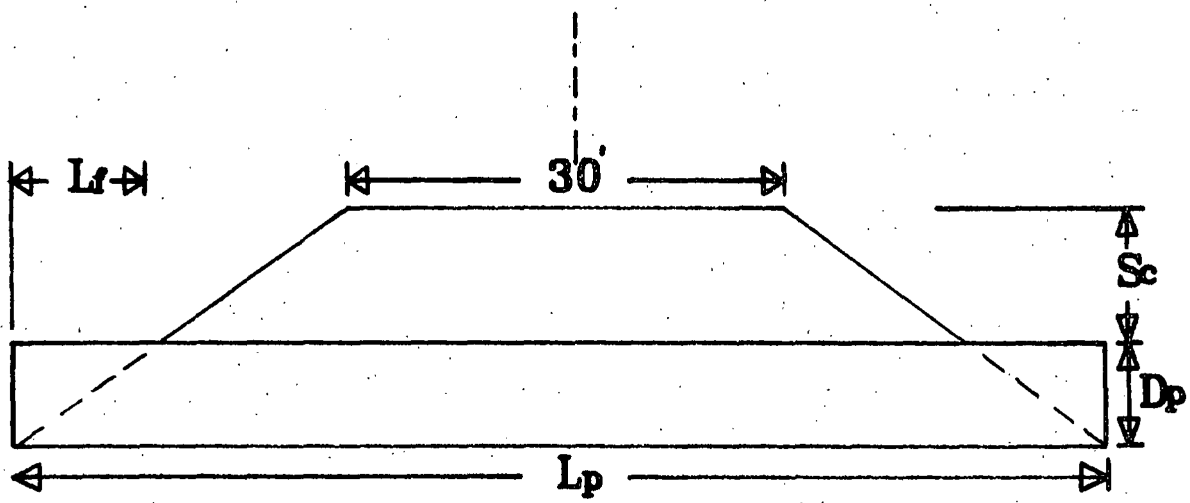
6) all vertical forces act on a horizontal plane that has a width equal to the pipe diameter;

7) roadway is 30 ft shoulder to shoulder; and

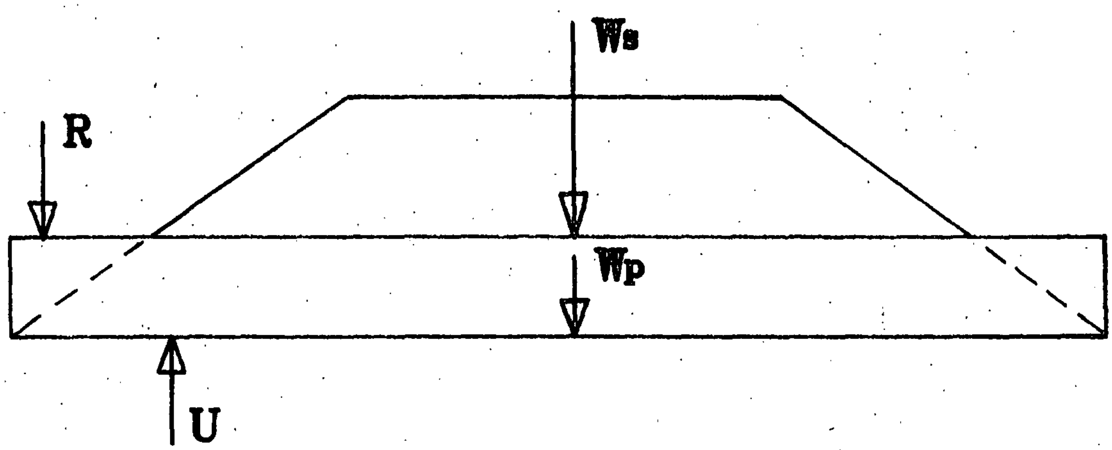
8) the soil slope extends from the bottom of the culvert to the edge of the shoulder and the pipe is not beveled.

Figure 5.1(a) is a schematic diagram used in the following static analyses. Based for the various analyses, the pipe length, L_f; Side slope; pipe diameter, D_p; depth of soil cover, S_c; and length of free pipe, L_f; may vary. The only thing constant throughout all analyses is the roadway width of 30 ft.

Figure 5.1(b) shows the forces used in the analyses where W_s is the weight of the soil cover, W_p is the weight of pipe, U is the



(a) Dimensions used in static analysis



(b) Forces assumed to be acting as a pipe

Fig. 5.1

uplift force from the porewater, 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 porewater pressure distribution at one third the distance, L_p. The location of R varies with the other loading conditions.

5.2. Moment Diagrams for Partial Flows

For this analysis, a 100 ft long culvert with 5 ft of fill is analyzed for zero flow, 50%, and 75% full flow. Because of constant pipe length, depth of soil fill, and roadway width, the side slope varies as the pipe diameter increases. For example, a 5 ft diameter pipe has a slope of 3.5 to 1 whereas a 9 ft diameter pipe has a side slope of 2.5 to 1. These dimensions are somewhat representative of the flexible pipe culverts in Iowa. Moments were taken about the upstream end of a cut section with the section increased by 1 ft increments from the upstream end. This approach allows a computation to determine the location and magnitude of the maximum moments.

The results of this analysis are contained Figures 5.2, 5.3, 5.4, 5.5, and 5.6. As shown in these graphs, the moments increase with increasing culvert diameter and decreasing water level in the culvert. The location of maximum moment shifts toward the centerline of the roadway as the pipe diameter increases. For example, at 50% full the maximum moment of a 4 ft pipe is at 32 ft from the inlet and at 47 ft. for a 12 ft pipe.

The required resisting force versus pipe diameter is shown in Figure 5.7. A 10 ft. diameter pipe that is totally plugged, with

Fig. 5.2 Moment along 4 ft. pipe from entrance

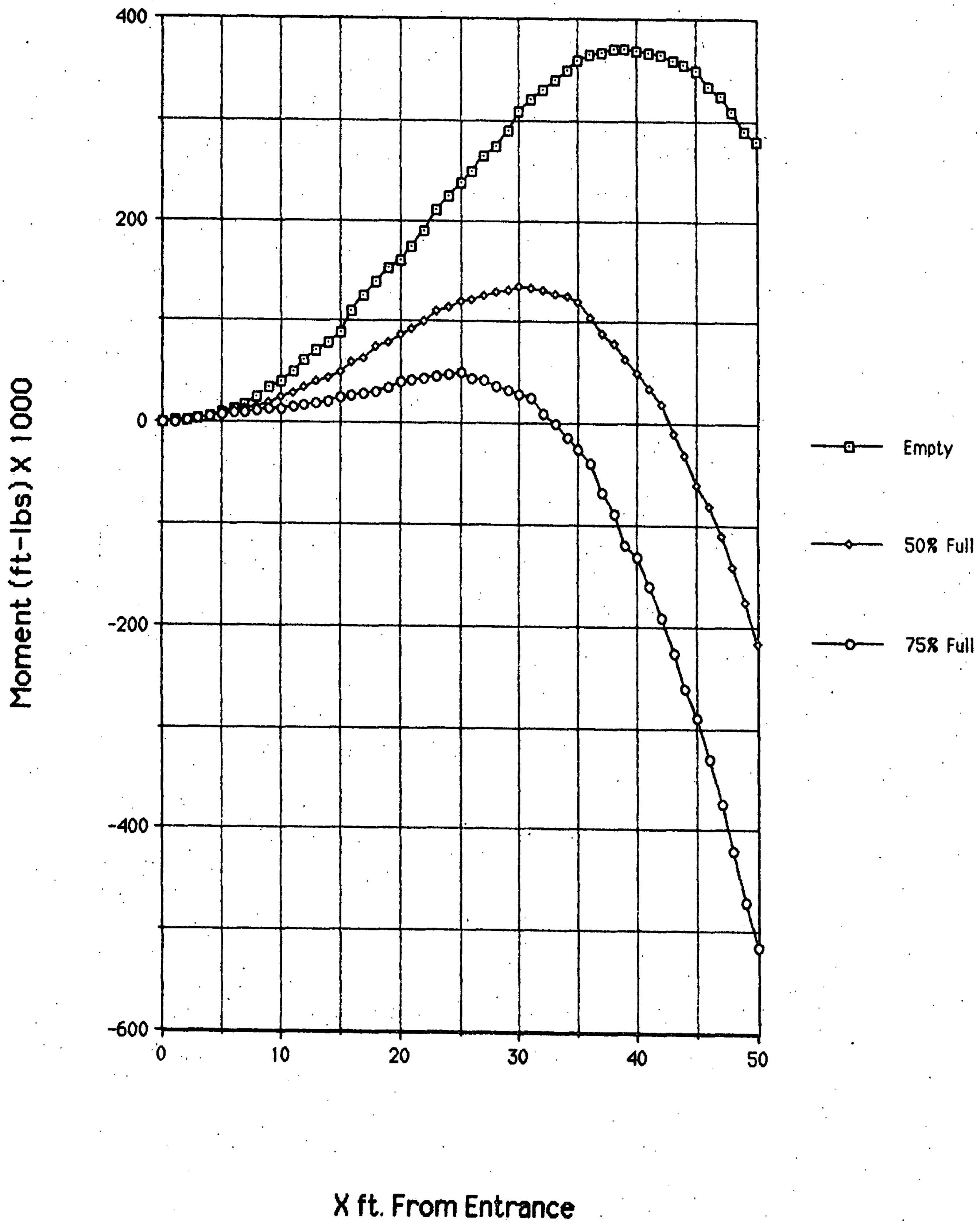


Fig. 5.3 Moment along 6 ft. pipe from entrance

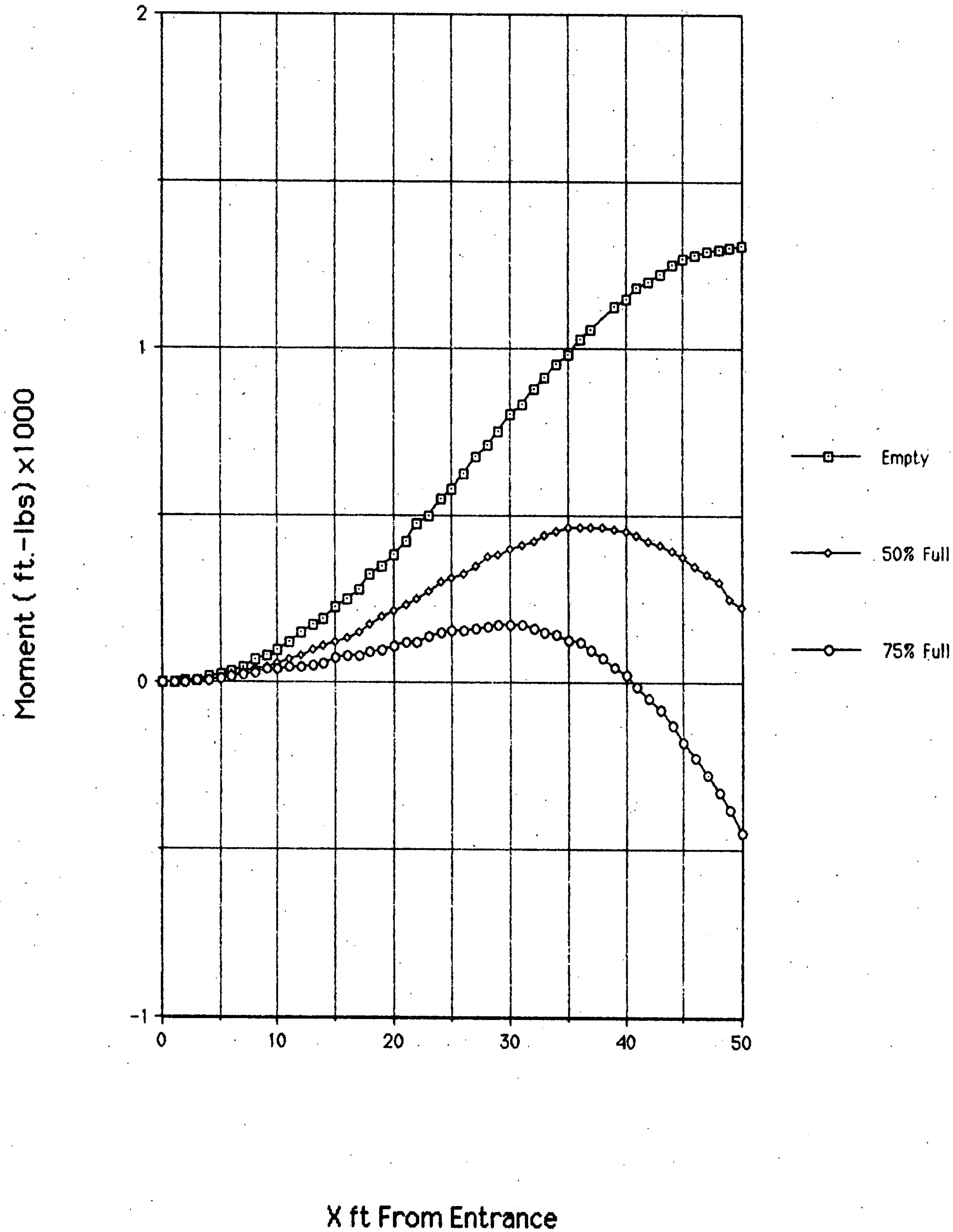


Fig. 5.4 Moment along 8 ft. pipe from entrace

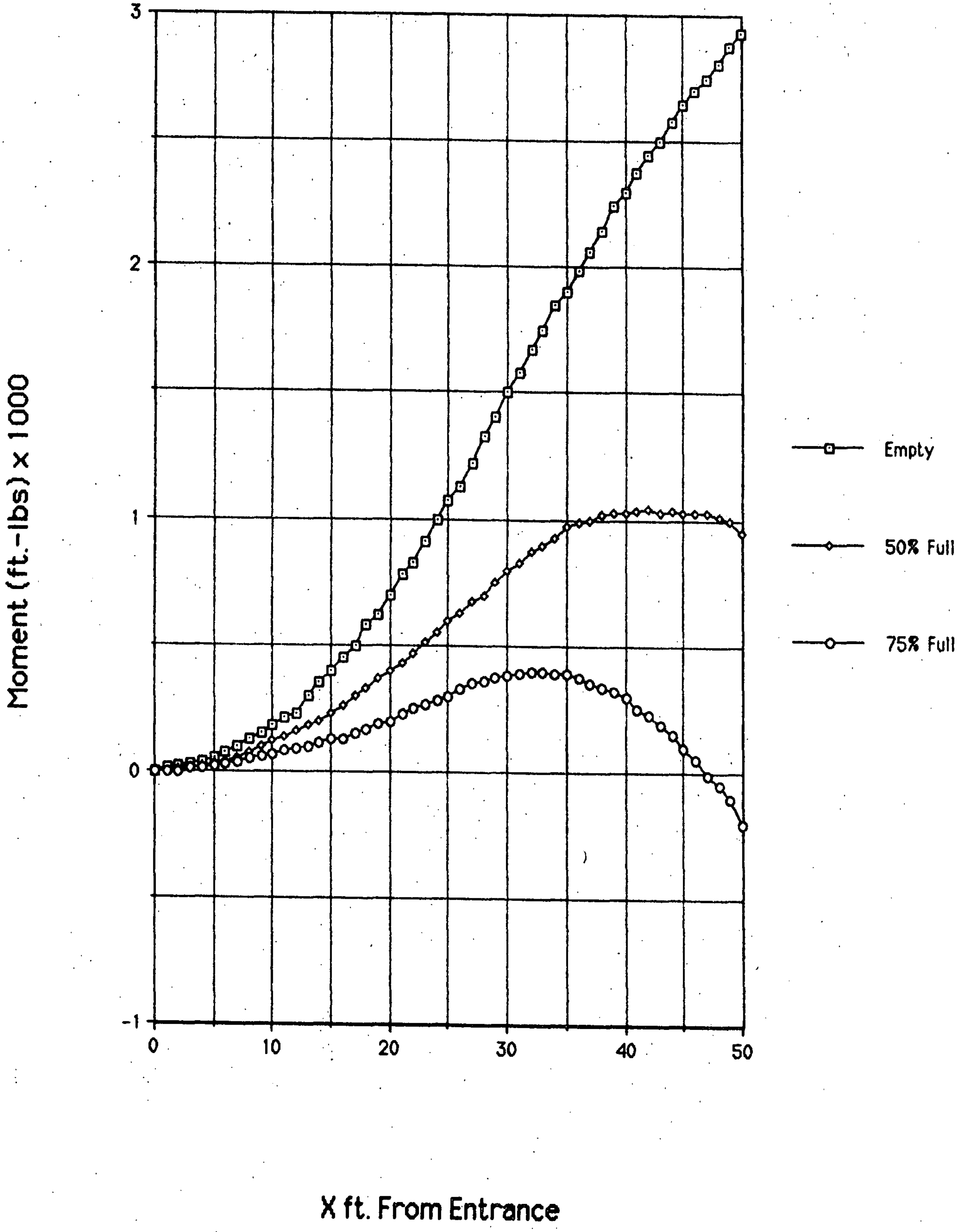


Fig. 5.5 Moment along 10ft. pipe from entrance

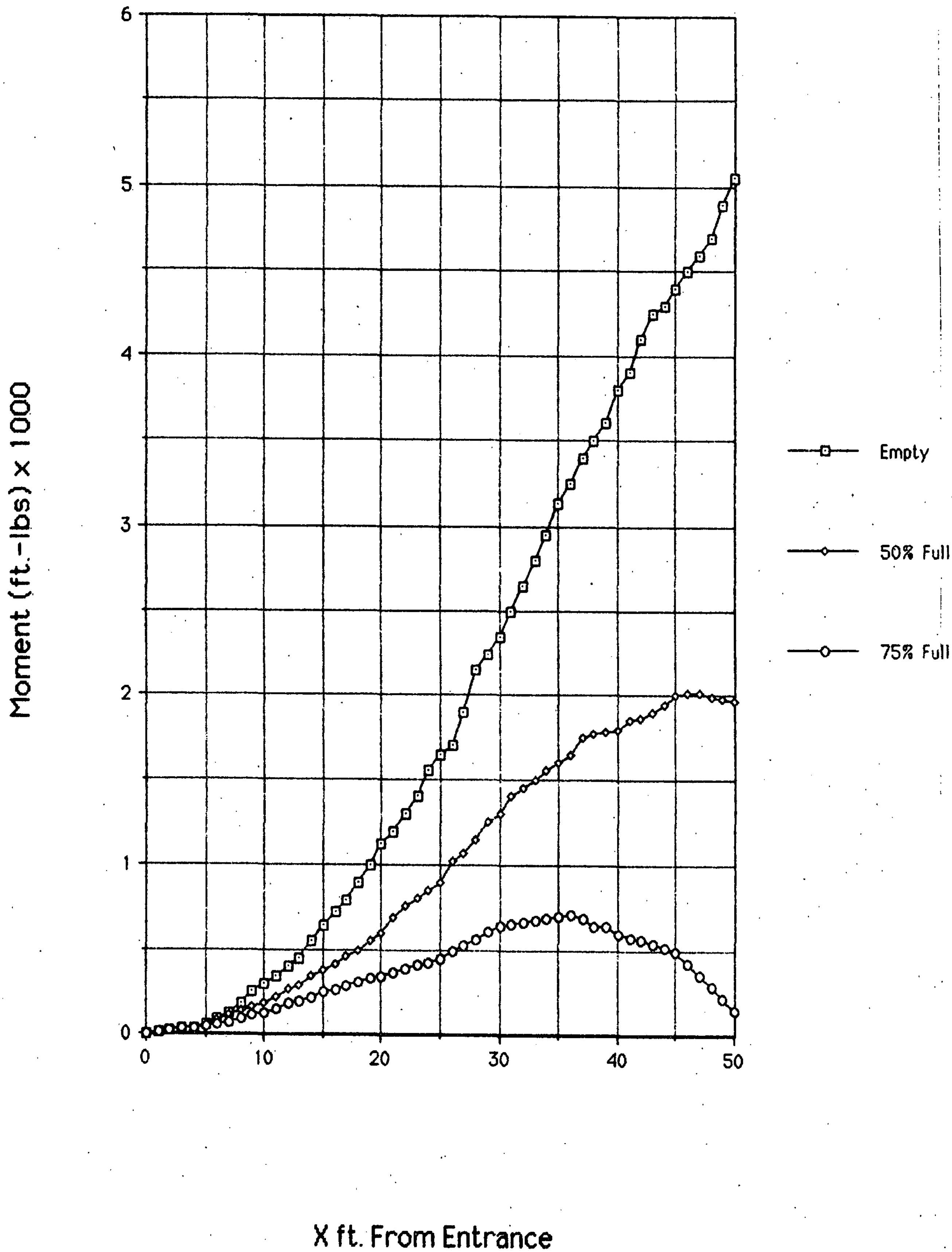


Fig. 5.6 Moment along 12 ft. pipe from entrance

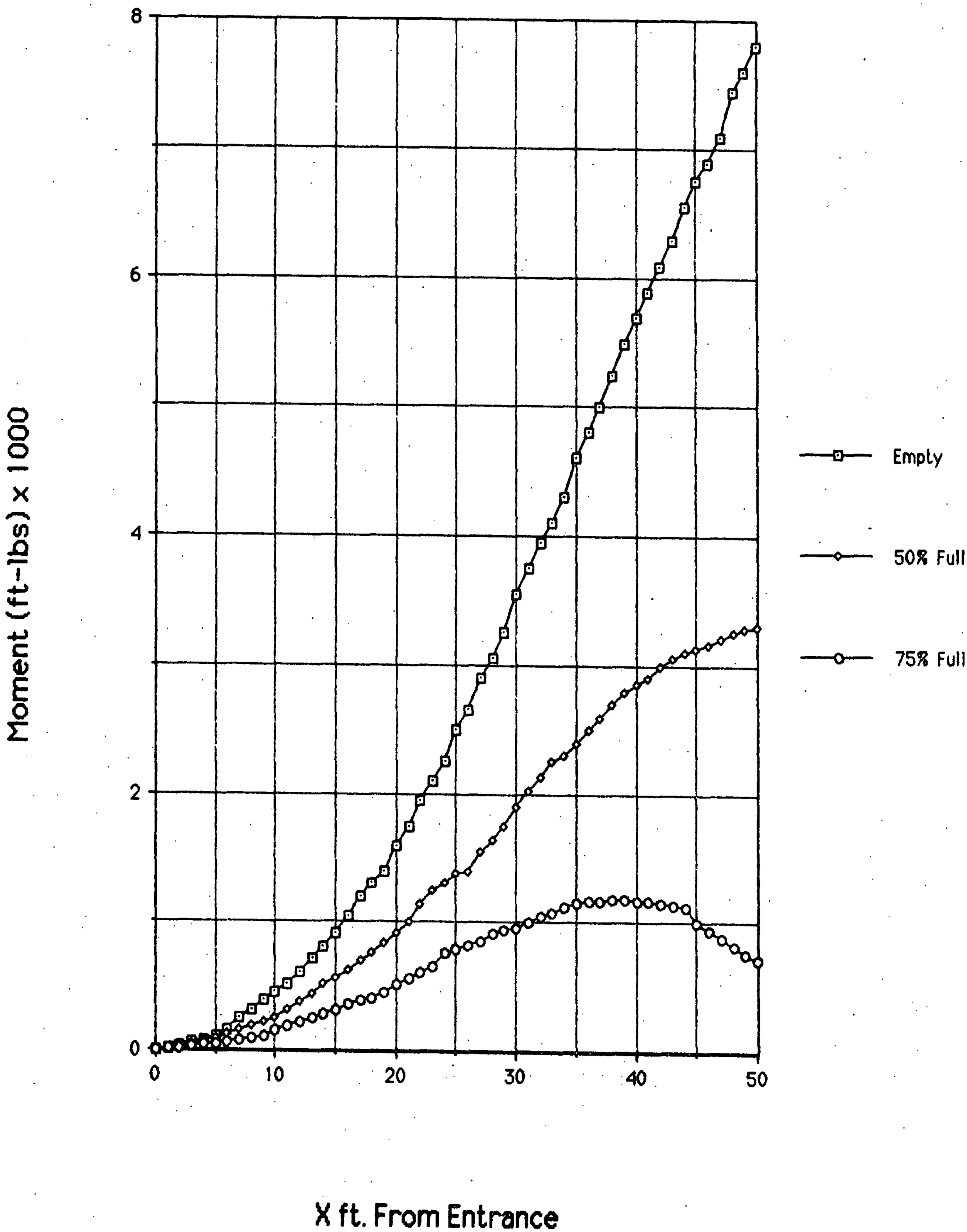
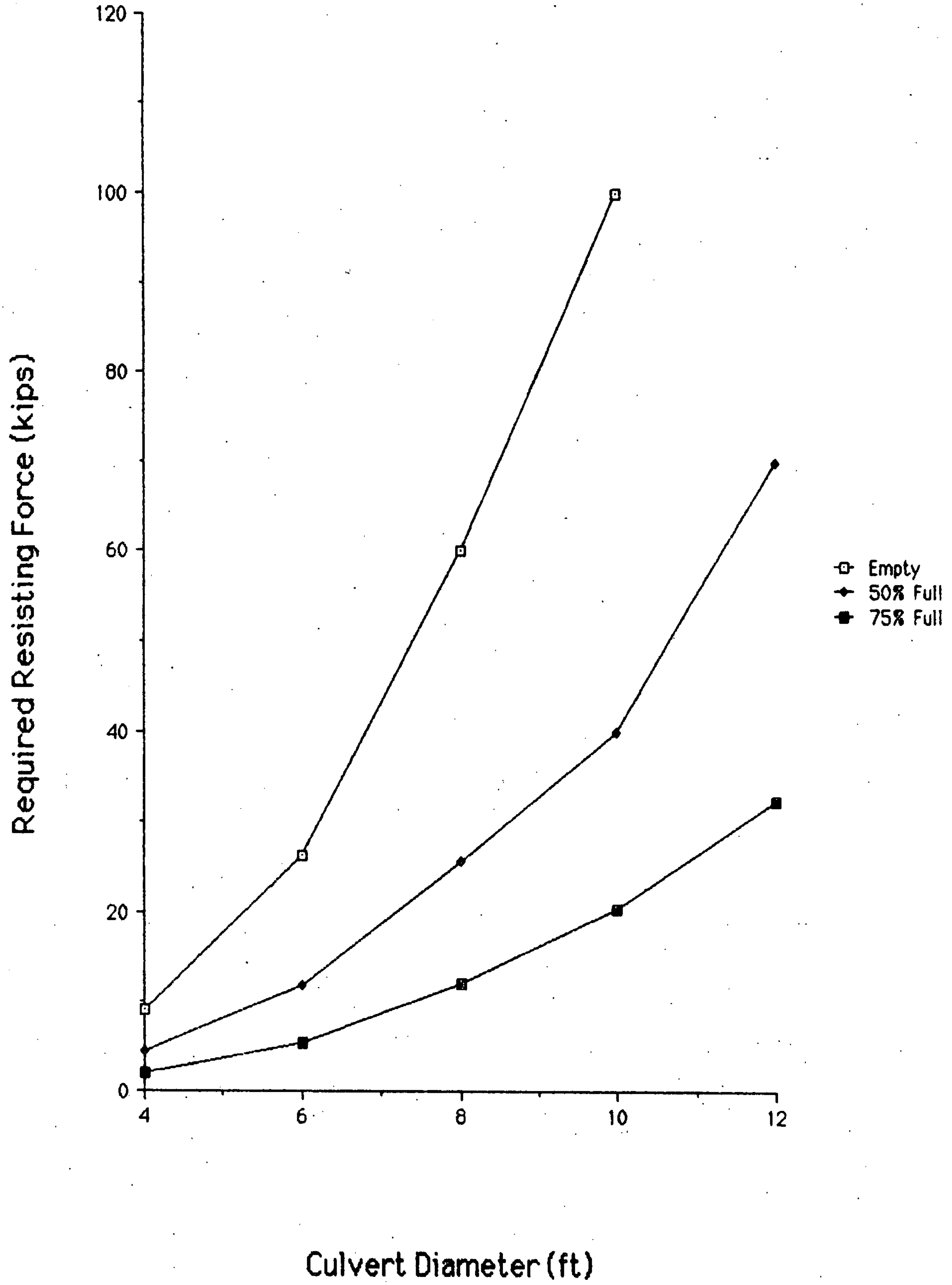


Fig. 5.7 Required Resisting Force vs. Pipe Diameter
Soil Cover = 5 ft and Slope varies with Pipe length



no flow, requires 100 kips to resist uplift. These same data are used in Table 5.1 to compute the volume of concrete needed in a headwall to resist uplift. In the case of a 4 ft diameter pipe that is plugged only 2.2 yd^3 of concrete are required whereas for a 12 ft diameter pipe over 25 yd^3 are needed.

Table 5.1. Mass and volume of concrete required to resist uplift on 100 ft long pipe with 5 ft of soil cover.

Pipe diameter (ft)	Slope	Flow conditions	Weight of concrete (lbs)	Volume of concrete (yd^3)
4	3.9:1	Empty	9000	2.2
		50% full	4333	1.0
		75% full	2000	0.5
6	3.2:1	Empty	26400	6.5
		50% full	11800	2.9
		75% full	5333	1.3
8	2.7:1	Empty	60000	14.8
		50% full	25581	6.3
		75% full	12100	3.0
10	2.3:1	Empty	100000	24.7
		50% full	40000	9.9
		75% full	20300	5.0
12	2.1:1	Empty	>100000	>25.0
		50% full	70000	17.3
		75% full	32400	8.0

When the assumptions of the previous analysis are compared to Iowa DOT standard road plans (RF-33, 1986 and RF-32, 1989, see Appendix C) it can be seen that the previous analysis is not conservative because the forgoing analysis assumed 5 ft of soil whereas the standard plans specify a minimum cover of 2 ft. In contrast, the volumes of concrete for pipes that are plugged are greater than those recommended by the FHWA (Notice N5040.3, 1974, see Appendix D). Table 5.2 shows the concrete volumes recommended by FHWA as well as those required by the Indiana DOT. The Indiana DOT (see Appendix E) indicated that their standards have been used for about 20 years and they are not aware of any uplift failure where these measures have been used. The rationale for the Indiana recommendations is unclear, but FHWA assumed buoyancy with no pore pressure dissipation.

Table 5.2. Volume of concrete for CMP as recommended by FHWA, Indiana DOT, and Iowa DOT

Pipe diameter (ft)	FHWA Concrete volume (yd^3)	Indiana DOT Concrete volume (yd^3)	Iowa DOT Concrete volume (yd^3)
4	1.5	2.4	2.5
6	4.1	3.6	3.8
8	6.8	4.8	5.4
10	12.4	6.2	7.1
12	20.1	7.7	9.2

5.3. Static Analysis with Variable Depth of Cover

A general static analysis was developed with constant road width and variable depth of soil cover. In this situation the pipe was treated as a free body with forces W_s, W_p, and U acting on it. This exercise allowed an interpretation of how much fill is required to withstand uplift assuming that the pipe is completely plugged. The results of this analysis are not intended for use in any design applications.

For one subset within this analysis, the slope was held constant and the pipe length varied as the depth of cover increased. The minimum depth of soil beneath the road that is required to balance the uplift force is called a critical soil cover. Figure 5.8 is a plot of pipe diameter versus the critical soil cover for slopes of 2:1, 4:1, and 6:1. These graphs show that the critical soil cover is relatively insensitive to slope with the ratio of critical soil cover to pipe diameter about 0.5. These data indicate that the 2 ft cover specified as minimum depths by the Iowa DOT standards will do little to resist uplift of pipes larger than 4 ft diameter and that some type of tie down on the upstream end of flexible metal culverts is required.

Recognizing that tiedowns are necessary, but that the resistance of the tiedown can be reduced by increasing soil cover, an analysis with a constant pipe length of 100 ft, variable soil cover and therefore variable side slope was conducted. The results of this analysis, shown in Figure 5.9, are not for design, but are intended to better understand the magnitude of the required forces.

Figure 5.9 implies that if the ratio of depth of soil cover to pipe diameter is about 0.73, no tiedowns are required. This ignores the flexibility of the corrugated metal pipe and presumes that the soil cover acts as a rigid continuum. Neither of these conditions are realistic and the first errs on the side of being too liberal because any free section of pipe, not covered by soil, may deflect as the result of high headwater pressures. However, on the whole,

Fig. 5.8 CRITICAL SOIL COVER vs. VARIABLE SLOPE at R. W. = 30 ft

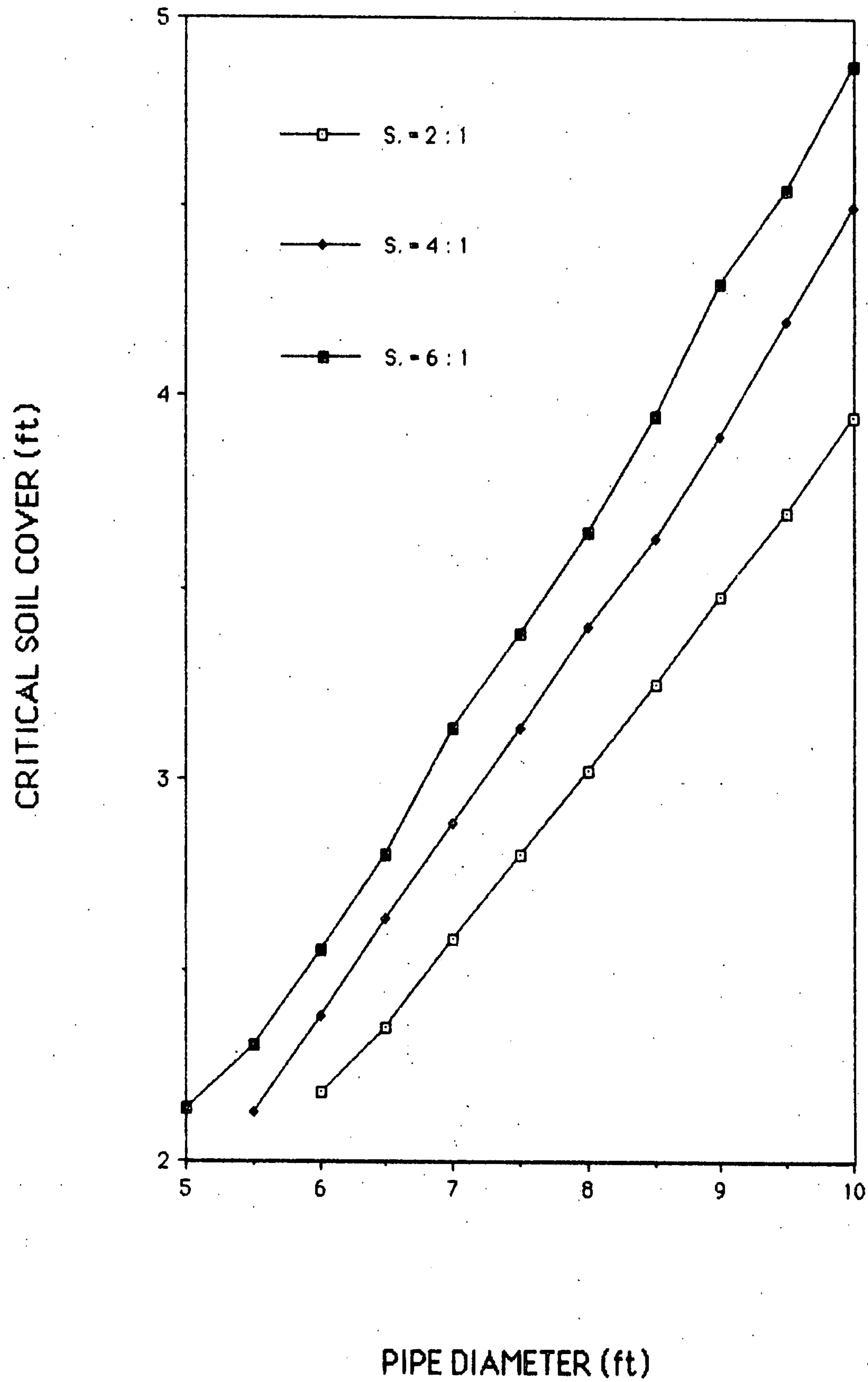
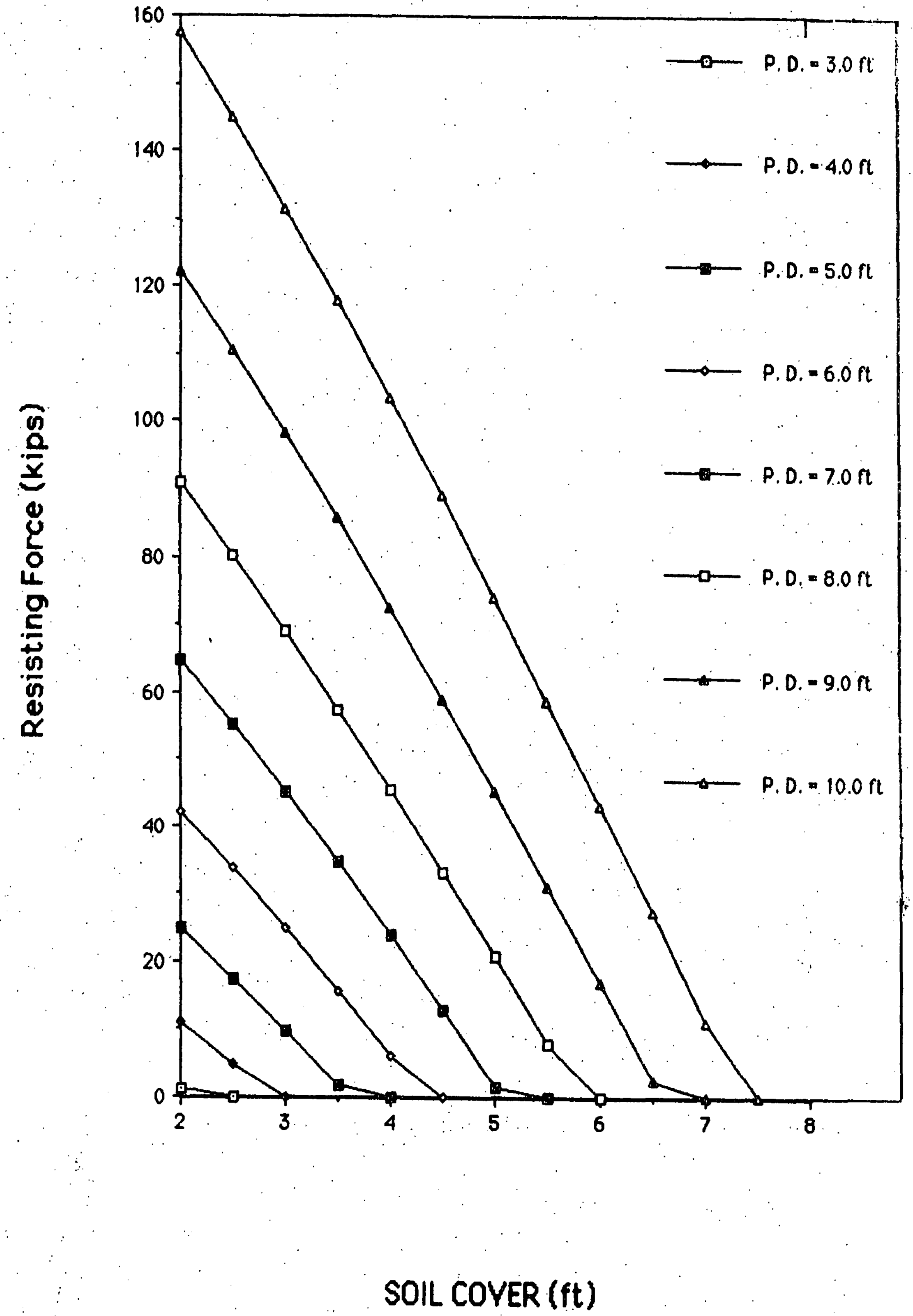


Fig. 5.9 SOIL COVER vs. MASS need at P. L. = 100 ft & R. W. = 30 ft



it is probably conservative to ignore some longitudinal stiffness in the CMP. The second assumption is conservative. Another conservative element in this analysis is the assumption that the pipe is completely plugged because the likelihood of complete pipe plugging decreases with increasing pipe diameter. The curves do show that the force required to prevent uplift decreases with increasing soil cover. For pipes between 6 and 10 ft in diameter, each foot of soil cover beneath the road reduces the required resistance by about 25 kips.

What is more relevant in Figure 5.9 is the force required for each size pipe with the specified minimum cover of 2 ft. These data are shown in Table 5.3 with the volume of concrete needed for this force. The concrete volumes in Table 5.1 are derived from the shear and moment diagram. The forces and concrete volumes in Table 5.3 are calculated from balancing the vertical forces. For comparison of the data based on the force balance, the volumes of

Table 5.3. Resisting Forces and Volumes of Concrete Required for plugged CM pipe (based upon assumptions described in section 5.3).

	2 ft Soil Cover		5 ft Soil Cover	
Pipe dia. (ft)	Resisting force (kips)	Concrete (yd_3)	Resisting force (kips)	Concrete (yd^3)
6	42	10.4	-	-
7	65	16.0	2	0.5
8	91	22.5	20	4.9
9	122	30.1	45	11.1
10	157	38.8	76	18.8

concrete for 5 ft of soil cover are also presented in Table 5.3. It can be seen that the volumes in Table 5.3 are much lower than those in Table 5.1. This results from balancing the forces rather than balancing the moments and indicates that the system is not in static equilibrium.

5.4. Discussion of Pore Pressure Analyses

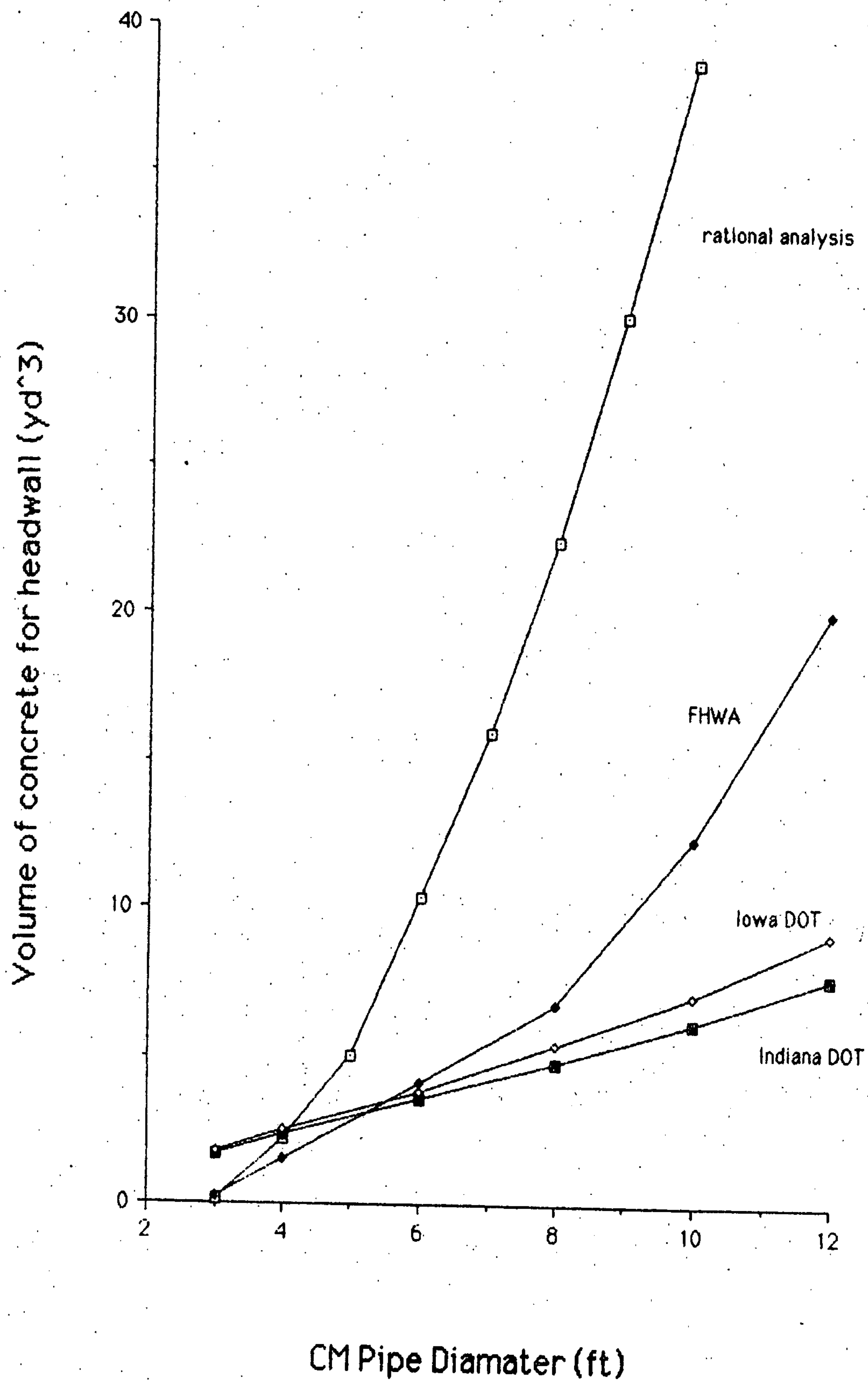
The static analyses of uplift forces on CMP indicate that for some geometries the soil, pipe, pore water systems are not in static equilibrium. The lack of equilibrium is caused by the symmetrical loading imposed by the pressures from soil cover and pipe and the unsymmetrical loading from the pore pressure distribution. It is also recognized that once the pipe starts to bend and/or the soil begins to deform, the assumptions of the static analysis are no longer valid. The static equilibrium could be restored by unsymmetrical loading imposed by: 1) the flow line of the water in the pipe, 2) a longitudinal bending resistance in the pipe that decreased downstream, or 3) soil deformation resistance that decreases with distance toward the centerline of the roadway. Chapter 2 of this report has shown that the flow line of the water in the pipe will be higher at the upstream end than at the tail water, consequently for certain flow conditions (but not all) the equilibrium can be obtained from loading 1 above. If the pipe is completely plugged, it is highly unlikely that either of the loadings suggested by 2 or 3 above would be realized; however, ignoring these resistance components adds considerable conservatism to this "rational analysis".

Although the Iowa DOT has no standards for tie downs on CMP, it has recommended headwalls that, in general, conform to the design shown in Figure 4.2. For this design, the resisting force and the required volume of concrete was calculated and the results included in Table 5.2.

The California DOT has in its bridge design specifications for CMP the statement: "Concrete headwalls or collars shall be placed at each inlet or outlet". California DOT does not provide an indication of what the magnitude of the resisting force should be.

The conservatism of the rational analysis in comparison with design practice of FHWA, Indiana DOT, and Iowa DOT is illustrated by Figure 5.10 where recommended concrete volumes for headwalls are plotted versus pipe diameter. The rational analysis gives results that are close to the recommendations of FHWA, Indiana DOT, and Iowa DOT for pipes less than 4 ft in diameter; but at larger diameters the required volumes are much larger. The Indiana DOT recommendations show the concrete volumes increasing linearly with increasing pipe diameter whereas the FHWA recommendations follow what appears to be an exponential type curve. This results in much less concrete required for larger pipes by Indiana than is required by FHWA. The Iowa recommendations fall between the FHWA and Indiana recommendations. The Iowa recommendations are based more upon judgement than upon a rational analysis and the basis for the Indiana recommendations is unknown. The FHWA recommendations are thought to be based upon a rational approach, but the assumptions for the analysis are not known. The large discrepancy between the

Fig. 5.10 Comparison of Volumes of Concrete required for headwall for tidedown on CMP



various design recommendations indicates that more study is required to obtain the best possible specifications for CMP design.

6. CASE HISTORIES

As part of the initial survey done on this project, several failure sites were selected for further investigation. Two such sites are presented.

6.1. Site 1

This site had a corrugated metal structural plate culvert installed in February, 1954. This culvert failed in June, 1976. The pipe was circular with a diameter of 12 feet and a beveled inlet (1 1/2 horizontal : 1 vertical). The pipe was 96 feet long on a slope of 3.81%. Figure 6.1 shows the profile of the pipe.

The profile at this site shows the average fill above the top of the pipe was 2.4 feet. The roadway was 28 feet wide with fore slope of 8 horizontal to 1 vertical at the inlet and 6 horizontal to 1 vertical at the outlet. Fill extended to the top of the beveled inlet. No information could be found on the highwater marks or discharge through the pipe during the storm that caused the failure. The pipe bent at about 26 feet from the pipe inlets and the inlet end of the pipe rose until it was at about the same elevation as the shoulder of the roadway. The pipe bottom collapsed inward beginning at about 22 feet upstream of centerline and extending to approximately the centerline of the road. The road grade washed out, but it was not clear whether water overtopped the road or undermined the pipe causing the embankment failure. No tie down structure, headwall, or cutoff wall was used

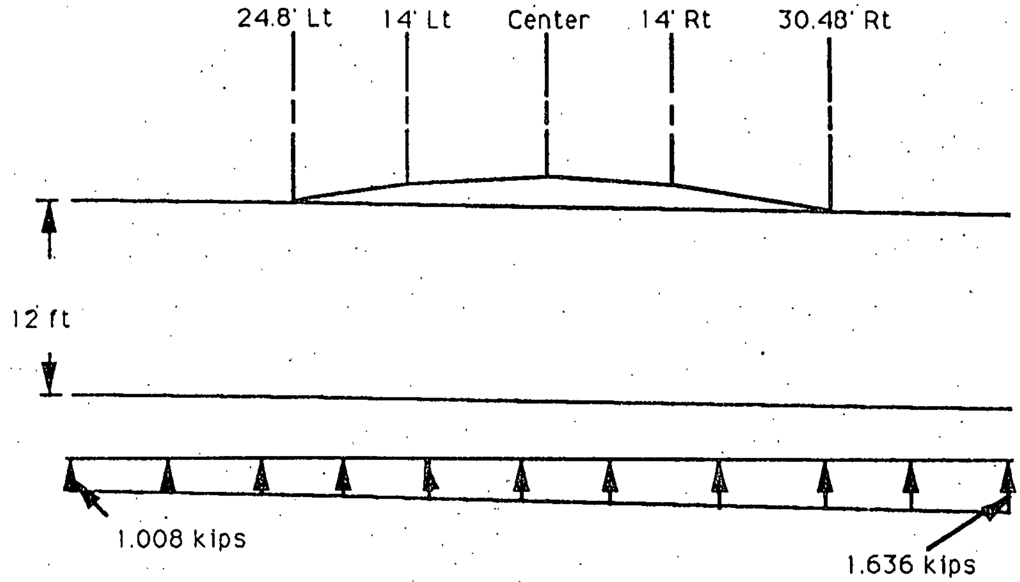
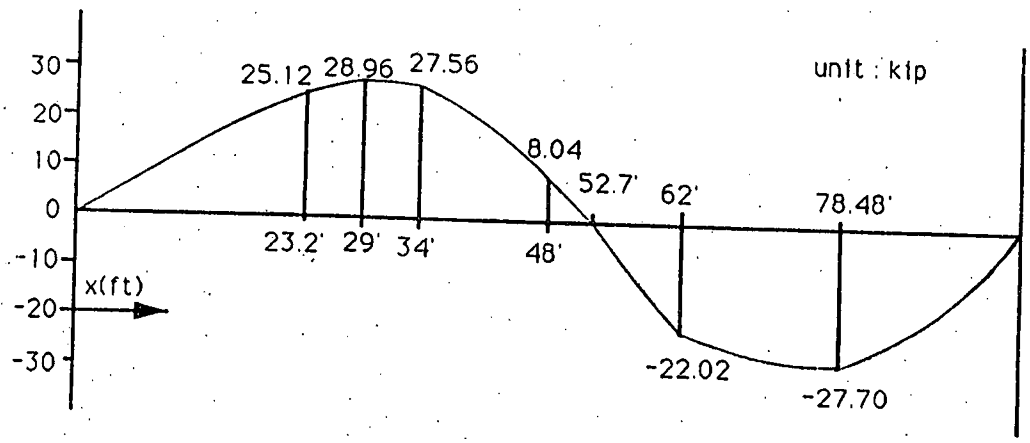
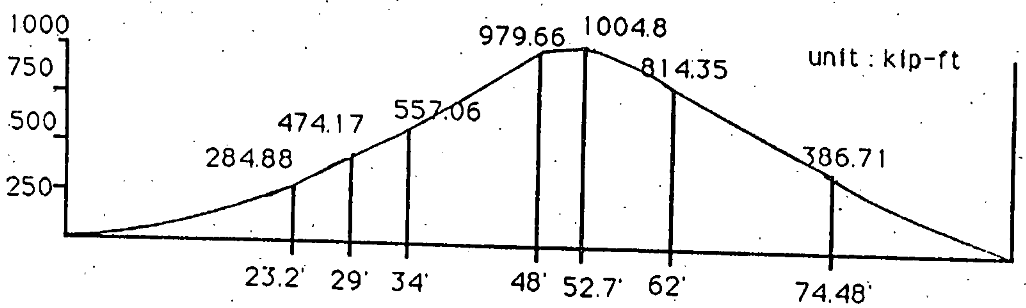


Fig. 6.1 Profile at Site 1



Shear Diagram



Moment Diagram

Fig 6.2 Shear and Moment diagram at Site 1 at this site nor was there any indication of diaphragms being used to control seepage along the pipe. From data at this site, it appears that the bend in the pipe occurred about 26 feet from the inlet end of the pipe.

The shear and moment diagrams for this site are shown in Figure 6.2. It was assumed based on theoretical considerations that the "bend" would occur at the point of maximum shear on the inlet end of the pipe. The shear diagram shows the maximum shear is about 29 kips and it is located at 29 feet from the inlet. This compares very well with the observed 26 feet form the inlet to the "bend" at this site. The moment that must be resisted at 29 feet is 474 kip-feet. This moment could be resisted by a mass of 16.9 kips (or 4.2 cubic yards of concrete) located at 1 foot from the inlet. The FHWA headwall design for a 12 feet diameter pipe would require about 20 cubic yards. The Indiana DOT and Iowa DOT designs would require 7 and 8 cubic yards of concrete, respectively. In this case, the calculated concrete mass was less than that required by either the Indiana or Iowa DOT standards.

6.2. Site 2

The pipe at this site was installed in July, 1976 and failed in September, 1986. The pipe was a structural plate CMP 10 feet in diameter with a projecting inlet. The pipe length was 120 feet. The roadway width was 28 feet with fore slopes of 2.5 horizontal to 1 vertical on both the inlet and outlet ends. The pipe slope was 3.67%. A seepage collar was placed about 20 feet downstream from the inlet (located at about the roadway shoulder). A tie down structure, composed of two wood piles driven beside the pipe with

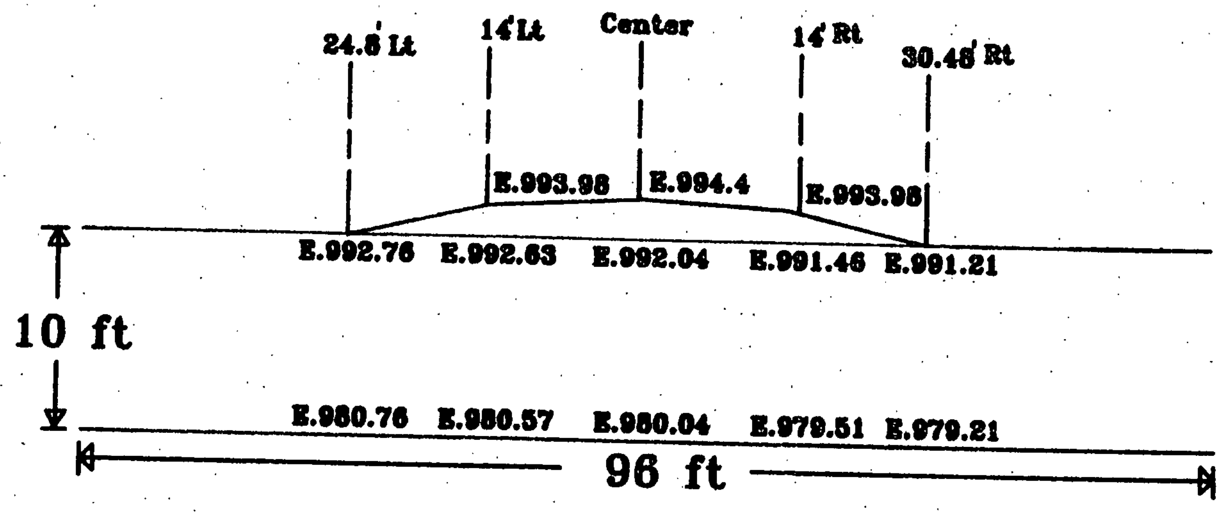


Fig. 6.3 Profile of Pipe at Site 2

two 3"x 16" wood planks across the top of the pipe and a 1/2" wire rope cable stretched across the pipe top, was constructed at this site.

Figure 6.3 shows the profile of the pipe at this site. The average fill above the top of the pipe was 2.8 feet at this site. No data on highwater levels or discharge could be obtained.

The inlet uplifted breaking the two wooden planks and stretching the cable. The road grade washed out and the entire culvert floated, moving downstream about 300 feet. Photos of this failure shows the seepage collar still in place. The bottom of the pipe, beginning near the bend, collapsed inward where there was only about 2 feet clearance at the top of the pipe. Although the pipe moved downstream, the pipe was not completely destroyed and much of the pipe could be used again.

The force required to cause the tie down structure to fail was estimated from the available data. The maximum bending stress of the wood plank was assumed to be 7.2 kips/in^2 (U.S.D.A., 1988). The moment of inertia was calculated to be 1372 in^4. Using the bending equation, the moment acting to cause failure is estimated at 118 ft-kips. This moment corresponds to a uniform load of 11.8 kips acting over a 5 feet length where the pipe and planks are in contact. In the photographs of the of the failed pipe, the indentation in the pipe made by the planks is clearly seen. Thus, the total force acting on the wood planks is estimated to be 59 kips. In addition to the wood planks, the stretch of the 1/2" rope cable must be incorporated. If the cable is assumed to be 1/2" in diameter (7 wires) with a yield stress of 250 kips/in^2, the ultimate load would be 36 kips. Information obtained from the county engineer indicated the cable was used not new cable. The estimated ultimate load was reduced by 20% to account for corrosion. Thus, the cables (2) would carry an additional load of 58 kips. The total estimated load in order to fail the tie down structure was estimated to be 117 kips.

For comparisons, the resisting force from Figure 5.9 for a 10' diameter pipe with 2.8 feet of cover is 140 kips. The calculated failure load was 117 kips. Because of the numerous assumptions that went into Figure 5.9 and the ultimate loading of the tie down structure, the agree is thought to be acceptable.

At this site a diaphragm was placed at about 20 feet from the inlet. Construction of these culverts normally call for over excavation and backfill with granular fill. If this procedure is used, the seepage collar (diaphragm) should as close to the inlet as possible and the granular fill should be isolated as much as possible from direct contact with the water at the inlet.

The analysis of the tie down structure would indicate the failure load would be about 100 kips, located at about one foot from the inlet. Table 5.3 shows that for a 10 ft. diameter plugged pipe with 2 ft. of cover, a resisting force of 157 kips would be required. From Figure 5.9 for a 10 ft diameter pipe with 2.8 feet of cover, the required resisting force is 140 kips. The analysis of Site 2 does not include any consideration of the resisting force generated because of the pipe stiffness. However, the general agreement between the required resisting force, as estimated by the pore water analysis (140- 157 kips), and the estimated tie down structure failure load (100 kips) suggests that the assumptions used to generate Figure 5.9 and Table 5.3 are reasonable.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1. General Conclusions

The following conclusions arise from this study:

1. A significant number of failures of CMP culverts are still occurring in Iowa despite design warnings issued in 1974.

2. Uplift failure seems to be the result of pore water pressure on the under side of the pipe.

3. There is a wide variation in the designs of tie downs structures being used.

4. The minimum size pipe that was found to have failed was a 72".

5. There were no special hydrologic, topographic, and geotechnical environments that appeared to be more susceptible to failures. However, most failures are thought to be at pipes flowing in inlet control.

7.2. Tentative Design Suggestions based on the Pore Water Analyses

The ultimate object of this research is to provide a rational basis for design of flexible metal culvert pipes against uplift. The immediate objective of this project is to clearly define the problem with regard to the magnitude of the forces involved; however in the interest of providing some immediate, practical results from this project it is suggested:

1. All CMP larger than 4 ft in diameter should be provided with headwalls or tiedowns at the upstream end of the pipe. For CMP greater than 6 ft tiedowns are essential.

2. The magnitude of the resisting force in the tiedown should be equivalent to no less than the weight provided by the volumes of concrete from FHWA recommendations shown in Figure 5.10. The current Iowa DOT design suggestions are less conservative, but probably adequate for most situations.

3. Resisting forces greater than the equivalent weight of concrete volume indicated by the "rational" curve in Figure 3.10 are likely to be over designed.

7.3. <u>Recommendations to Develop Rational Design of Tiedowns for CMP</u>

Although some states and FHWA have criteria for CMP tiedowns, it is apparent that these criteria are not based upon consistent theories. In order to arrive design standards for well engineered tiedown structures for flexible metal pipe culverts, a rational design process should be developed. The pore water analysis can be improved by combining the flow line analysis of Chapter 4 with and improved version of the pore water force analysis in Chapter 5. The pore water force analysis can be improved by obtaining experimental data on the longitudinal flexural strength of large diameter CMP. A second component to improve the pore water analysis is to experimentally and theoretically develop equations to characterize the stress strain characteristics of the soil in the embankment above and adjacent to the pipe.

7.4. <u>Recommended Action</u>

All current and future CMP culverts above 4 ft. in diameter should have adequate tie downs to resist uplift forces. Concrete headwalls or slope collars are recommended for most locations; however, other engineered tie down structures can be successful if they provide adequate resistent force as outlined in Chapter 5. Adequate attention should be given to spacing of the bolts connecting the pipe to the concrete mass. The concrete should extend at least 4 feet below the pipe and to the side of the pipe about 1/4 the pipe diameter.

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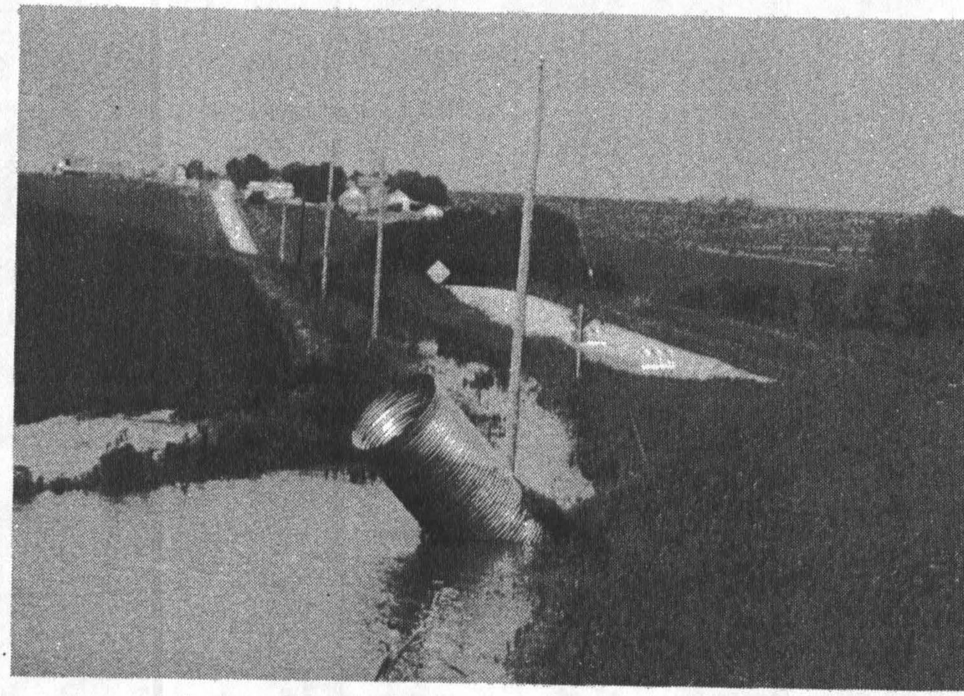
9. California Department of Transportation. "Highway Design Manual", pages 820-3 to 820-11, August 5, 1988.

APPENDIX A

PHOTOS OF CULVERT FAILURES

Figure A-1. Corrugated metal pipe failure, 78" diameter, failure occurred in summer, 1989 in southwest Iowa. This photograph shows the inlet flotation.

Figure A-2. Photograph from outlet end of the pipe from Figure A-1. Note the bottom of the pipe has collapsed inward in a V-shape. (78" diameter)



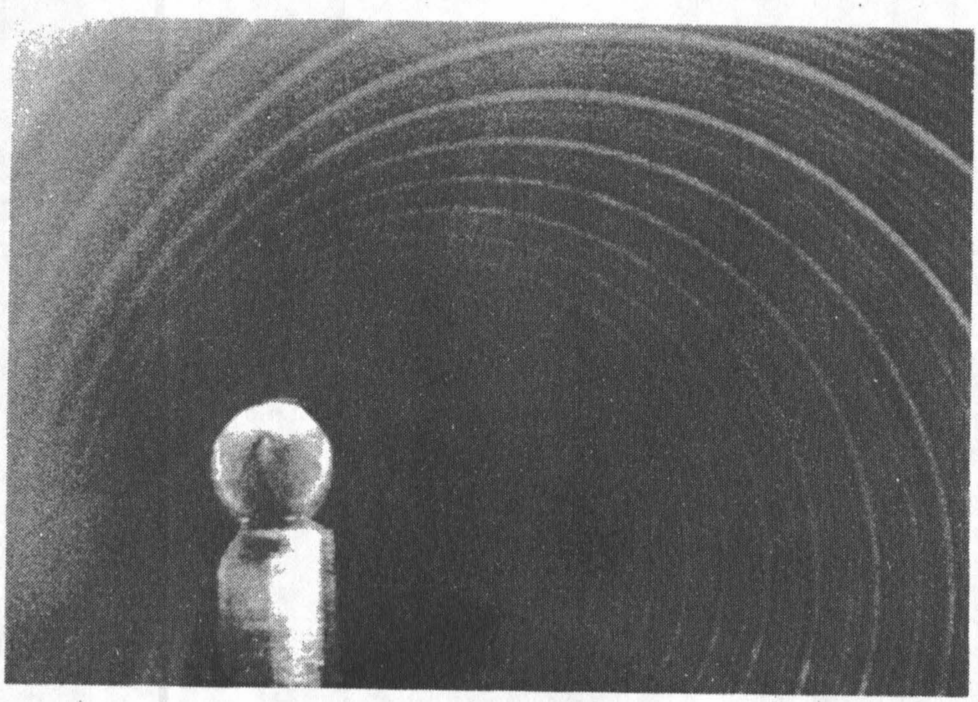


Figure A-3. Collapse of the corrugations at the top of this 120" diameter pipe caused the stretching of corrugations and the collapse of the bottom inward. (Photograph from Darrel Coy, Iowa DOT)

Figure A-4. Remains of a pile tie down structure after failure. The pipe was 144" in diameter and floated downstream. Note the shear failure of the two 3" by 16" bridge timbers. (Photograph from Darrel Coy, Iowa DOT)

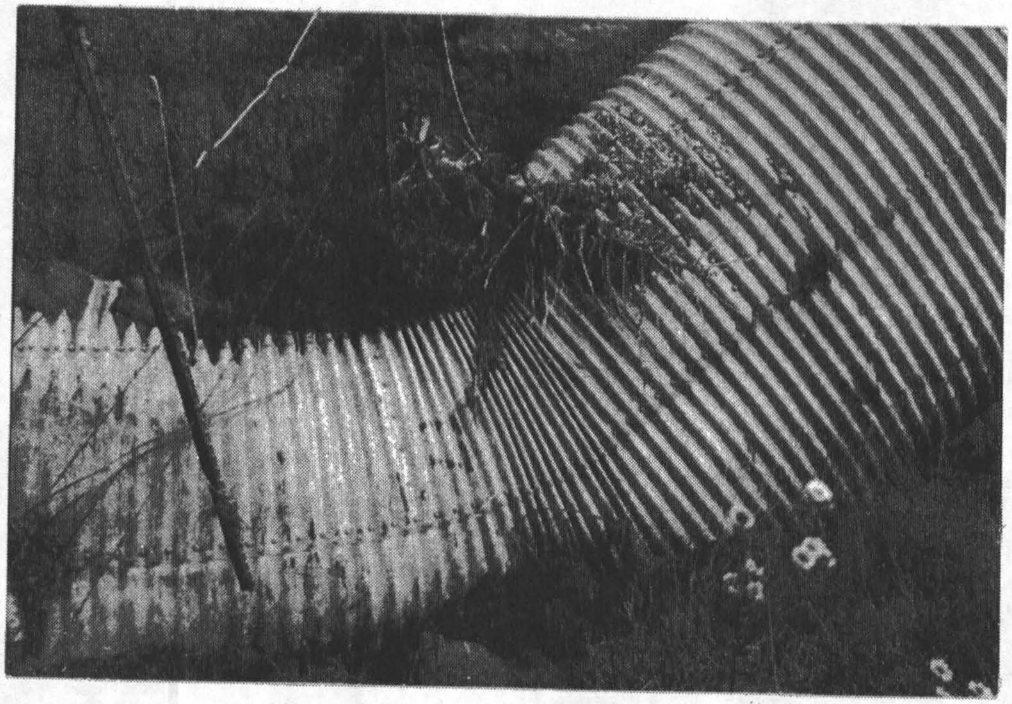


Figure A-5. View from the centerline of road showing 120" diameter pipe, about 300 feet long after failure. (Photograph from Darrel Coy, Iowa DOT)

Figure A-6. View of 120" diameter pipe after failure. Note the bottom of pipe is collapsed inward. (Photograph by Darrel Coy, Iowa DOT).

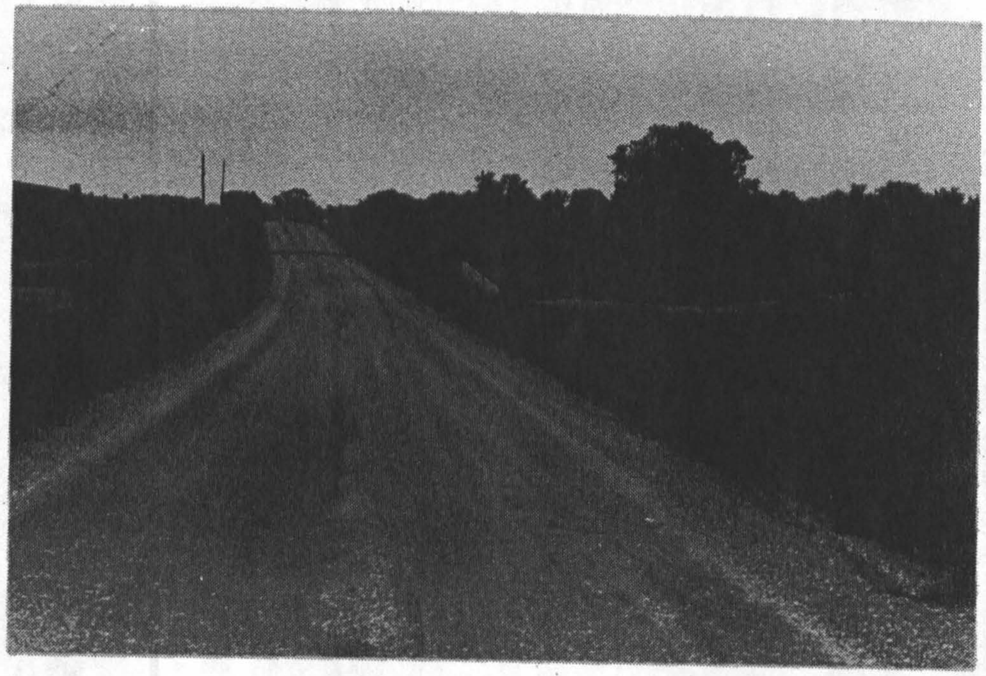
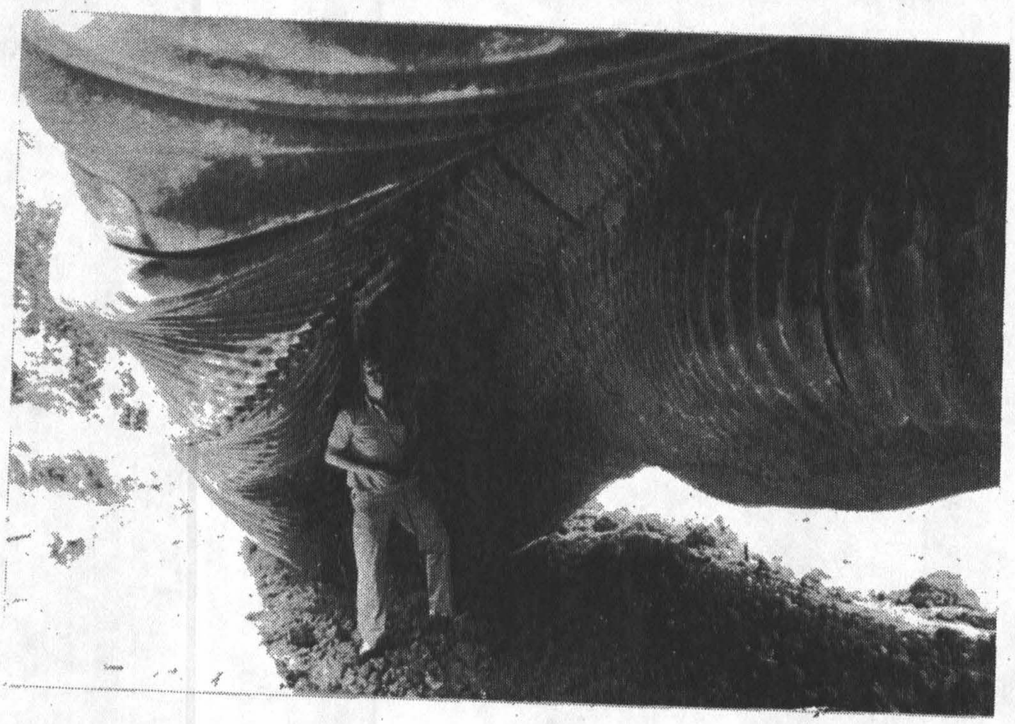


Figure A-7. View of bottom of corrugated metal pipe culvert showing collapse inward. The pipe was 144" in diameter. The person is standing about 20 feet from the pipe inlet. (Photograph from Darrel Coy, Iowa DOT).

Figure A-8. Photograph inside the pipe (144" diameter) about 25 feet from the outlet looking upstream. This view shows the collapse of the bottom inward. (Photograph be Darrel Coy, Iowa DOT).



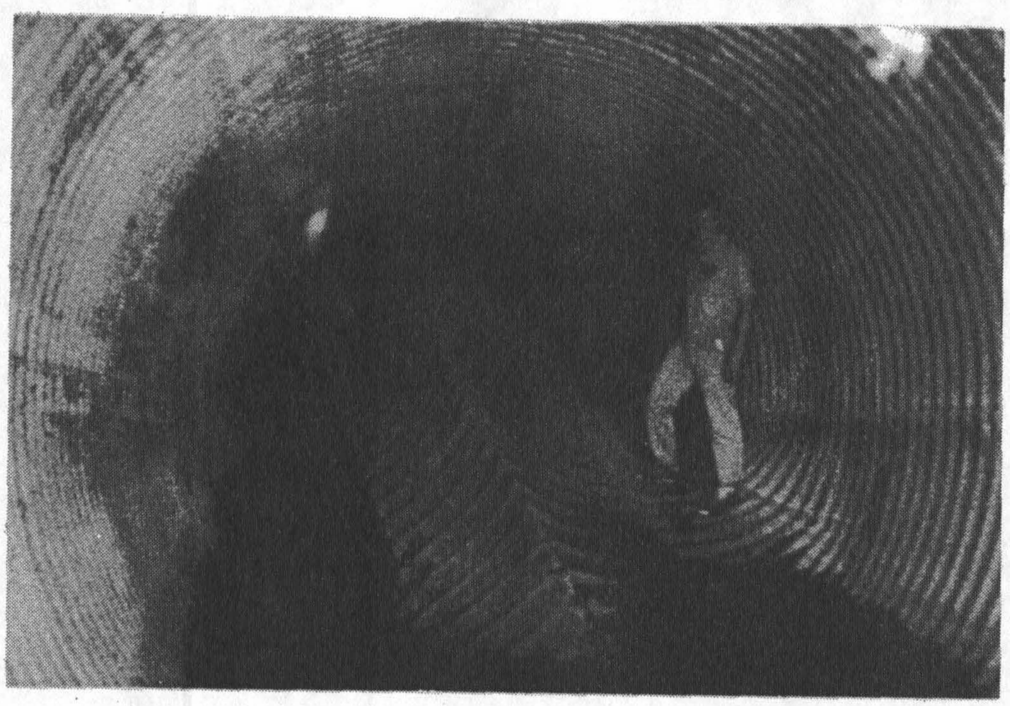


Figure A-9. Photograph showing failure of pile tie down structure. This failure pulled the piles out of the ground as the inlet floated. The pipe was 120" diameter. The failure occurred in summer, 1989.

Figure A-10. Inlet view of corrugated metal pipe culvert inlet. The pile tie down structure was pulled from the ground. Note the 1/2" wire rope cable connecting the piles. The pipe diameter is 120".