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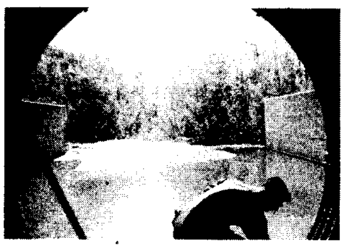

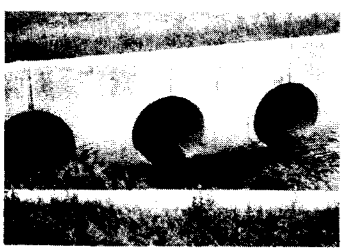

TECHNICAL REPORT GL-88-2

LIFE CYCLE COST FOR DRAINAGE STRUCTURES

by

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February 1988
Final Report

Prepared for DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20314-1000

GEOTECHNICAL LABORATORY

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1a. REPORT SECURITY CLASSIFICATION Unclassified	1b. RESTRICTIVE MARKINGS
2a. SECURITY CLASSIFICATION AUTHORITY	3. DISTRIBUTION/AVAILABILITY OF REPORT Approved for public release; distribution unlimited.
2b. DECLASSIFICATION/DOWNGRADING SCHEDULE	
4. PERFORMING ORGANIZATION REPORT NUMBER(S) Technical Report GL-88-2	5. MONITORING ORGANIZATION REPORT NUMBER(S)
6a. NAME OF PERFORMING ORGANIZATION USAEWES Geotechnical Laboratory 6b. OFFICE SYMBOL (If applicable) CEWES-GP-EM	7a. NAME OF MONITORING ORGANIZATION
6c. ADDRESS (City, State, and ZIP Code) PO Box 631 Vicksburg, MS 39180-0631	7b. ADDRESS (City, State, and ZIP Code)
8a. NAME OF FUNDING/SPONSORING ORGANIZATION US Army Corps of Engineers 8b. OFFICE SYMBOL (If applicable) CEEC-EG	9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER
8c. ADDRESS (City, State, and ZIP Code) 20 Massachusetts Avenue Washington, DC 20314-1000	10. SOURCE OF FUNDING NUMBERS PROGRAM ELEMENT NO. / PROJECT NO. / TASK NO. / WORK UNIT ACCESSION NO.

11. TITLE (Include Security Classification)
Life Cycle Cost for Drainage Structures

12. PERSONAL AUTHOR(S)
Potter, John C.

13a. TYPE OF REPORT	13b. TIME COVERED	14. DATE OF REPORT (Year, Month, Day)	15. PAGE COUNT
Final report	FROM Nov 85 TO Sep 87	February 1988	72

16. SUPPLEMENTARY NOTATION
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

17. COSATI CODES			18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number)
FIELD	GROUP	SUB-GROUP	Drainage; Pipe
			Durability; Service life
			Life cycle cost

19. ABSTRACT (Continue on reverse if necessary and identify by block number)

Principal factors involved in the design of drainage structures include hydrology, soil conditions, material strength, material durability, cost, and type of facility being drained. Although not necessarily overriding, the cost is often one of the most important factors. This cost should be the total, overall cost of the alternative over its projected life or life cycle cost (LCC). Unless the LCC is considered over first cost, the owner cannot be assured of receiving maximum value for his construction and maintenance dollars. Except for determining a service life for the various types (materials) of drainage structures, the procedures for LCC analysis are well established. LCC based economic studies are an integral part of the complete design process and are a requirement specified in Technical Manual 5-802-1. AR 11-28/AFR 178-1 gives the basic criteria and standards for economic studies by and for the Departments of the Army and Air Force. The

(Continued)

20. DISTRIBUTION/AVAILABILITY OF ABSTRACT ☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT. ☐ DTIC USERS	21. ABSTRACT SECURITY CLASSIFICATION Unclassified
22a. NAME OF RESPONSIBLE INDIVIDUAL	22b. TELEPHONE (Include Area Code) / 22c. OFFICE SYMBOL

19. ABSTRACT (Continued).

guidelines presented in Part II of this report can be used to estimate the service life of a particular design or ensure a 50-year service life. Thus, the procedures for economic analysis described in Technical Manual 5-802-1 can be used to determine LCC. The alternatives can then be order ranked based on LCC, and the best design can be rationally and confidently selected.

PREFACE

The investigation reported herein was sponsored by the Office, Chief of Engineers (OCE), under the work effort "Life Cycle Cost for Drainage Structures (Pipe)," of the Facilities Investigation and Studies Program. The OCE Technical Monitor was Mr. Edwin Dudka.

The study was conducted at the US Army Engineer Waterways Experiment Station (WES) from November 1985 through September 1987 by the Pavement Systems Division (PSD) of the WES Geotechnical Laboratory (GL).

The research was conducted and the report was written by Dr. J. C. Potter, PSD. The study was under the supervision of Messrs. H. H. Ulery, Jr., Chief, PSD; H. Green, Chief, Engineering Analysis Group, PSD; and D. M. Ladd, Chief, Criteria Development Unit, PSD. The work was conducted under the general supervision of Dr. W. F. Marcuson III, Chief, GL. The report was edited by Ms. Odell F. Allen, Information Products Division, Information Technology Laboratory.

Comments on the durability guidelines were solicited from industry associations and corporations. Specifically, the guidelines for metal pipe were sent to the National Corrugated Steel Pipe Association, the Aluminum Association, and Armco. The concrete pipe guidelines were sent to the American Concrete Pipe Association. Unibell, the polyvinyl chloride (PVC) pipe manufacturers association, the Plastics Pipe Institute, the polyethylene pipe manufacturers association, and Advanced Drainage Systems (ADS) Inc., a major polyethylene pipe manufacturer, received copies of the plastic pipe guidelines. The National Clay Pipe Institute was consulted on clay pipe durability.

Responses were received from both associations and corporations. Comments on metal pipe durability came from the National Corrugated Steel Pipe Association, the Aluminum Association, Armco, Bethlehem Steel Corporation, Pacific Corrugated Pipe Co., Lane Enterprises, Caldwell Culvert Company, and Dow Chemical. The American Concrete Pipe Association, the Ohio Concrete Pipe Manufacturers' Association, and the Ohio Department of Transportation commented on the concrete pipe guidelines. The plastic pipe design procedure described in this report is based on the industry-consensus of American Association of State Highway and Transportation Officials (AASHTO) proposed specifications, to which Unibell, the Plastics Pipe Institute and ADS have contributed. Comments were also received separately from Unibell, Contech, the Plastics Pipe Institute and ADS. The National Clay Pipe Institute has concurred with the clay pipe durability guidelines.

The responses noted above were studied with additional research as necessary to resolve conflicts. The guidelines were then revised as appropriate.

COL Dwayne G. Lee, CE, was the Commander and Director of WES during the preparation and publication of this report. Dr. Robert W. Whalin was Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
inches	2.54	centimetres
kips (force)	4.448222	kilonewtons
miles (US statute)	1.609347	kilometres
pounds (force) per square inch	6.894757	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square inches	6.4516	square centimetres

LIFE CYCLE COST FOR DRAINAGE STRUCTURES

PART I: INTRODUCTION

Background

1. Many factors are involved in the design of drainage systems. Principal factors are hydrology, soil conditions, material strength, material durability, cost, and type of facility or site being drained. While not necessarily overriding, the cost is often one of the most important factors. This cost should be the total, overall cost of the alternative over its projected life, or life cycle cost (LCC). Unless the life cycle cost is considered over first cost, the owner cannot be assured of receiving maximum value for his construction and maintenance dollars. LCC based economic studies are an integral part of the complete design process and are a requirement specified by Technical Manual 5-802-1. AR 11-28/AFR 178-1 gives the basic criteria and standards for economic studies by and for the Departments of the Army and Air Force.

Purpose

2. The purpose of this report is to provide supplemental guidance in performing LCC studies to determine the relative economic ranking of alternative drainage systems using pipes fabricated from various construction materials. Current Corps of Engineers design criteria do not include guidance for estimating the expected service life for drainage structures. Therefore, guidance in determining the appropriate service life for a particular design alternative is included in addition to the supplemental guidance on economic calculations.

Scope

3. This report provides supplemental information required to perform LCC analyses of military drainage structures to determine the relative economic rating of design alternatives. Methods of estimating service life or

ensuring a particular design life are also given for the more common pipe materials used in drainage structures. Metal, concrete, clay, and plastic pipe durability guidelines are provided, including procedures for estimating the service life of steel and concrete pipes.

PART II: SERVICE LIFE GUIDELINES

General

4. The most difficult and controversial aspect of life-cycle cost analysis (LCCA) for drainage structures is establishing a service life for each material type. Service life is a function of pipe material, the environment in which it is installed, and the effect of additional measures taken to protect the pipe from deterioration. Service life is also subject to biased estimation by investigators working in particularly harsh or mild environments and by some vendors and trade associations. LCCA requires a realistic estimate of service life. So, currently available performance data and durability guidelines from various sources outside of the Corps of Engineers have been collected, analyzed, and synthesized into a comprehensive but uncomplicated procedure. Guidelines for predicting a service life or ensuring a particular design service life for the more common types of pipes found in drainage structures were developed and included in this report.

Metal Pipe

5. Metal pipe performance data and durability models based on this performance can be found in the "Handbook of Steel Drainage and Highway Construction Products," (American Iron and Steel Institute (AISI) 1983), "Durability of Drainage Pipe," (Transportation Research Board 1978), and various technical papers such as those found in "Symposium on Durability of Culverts and Storm Drains" (Transportation Research Board 1984) and those listed in the bibliography.

6. The information contained in these resources has been synthesized into a flexible and coherent durability guide, consisting of two sections. The first section is a set of guidelines which establish environmental limits for satisfactory performance of metal pipe for at least 50 years. These guidelines encompass the majority of drainage applications. For applications in environments outside of these limits or when a service life of other than 50 years is desired, a second section is provided. Using this section, a combination of metal pipe and protective coatings can be designed to give a wide variety of service lives over a wide range of environmental conditions.

7. For a design service life of 50 years, the environmental limits for metal pipe that have been synthesized from a literature review are as follows:

Type of Material Used to Make Pipe	Soil and Water pH	Minimum Soil Resistivity ohm-cm
Galvanized Steel (AASHTO M218)	6 - 8.0	≧ 2,500
Aluminized Steel, Type 2 (AASHTO M274)	5 - 9.0	≧ 1,500
Aluminum (Alclad 3004)	5 - 9.0	≧ 1,500
	or 5.5 - 8.5	≧ 500
Stainless Steel, Type AISI 409	5 - 9.0	≧ 1,500
Cast Iron	6 - 9.0	≧ 1,500

8. These limits apply to pipes of adequate structural design as determined by an accepted procedure such as that presented in the "Handbook of Steel Drainage and Highway Construction Products" (AISI 1983) without the benefit of additional, sacrificial thickness. Also, stainless steel (Type 409) may be used to carry acid coal mine water, without regard to pH, because of the particular chemistry of acid coal mine water.

9. The limits given in the "Handbook of Steel Drainage and Highway Construction Products" (American Iron and Steel Institute 1983) and "Corrugated Metal Pipe Durability Guidelines" (Federal Highway Administration 1979) are somewhat broader, but they are not based on a specific design life.

10. For conditions outside of the above limits, or a design life other than 50 years, a more sophisticated approach is required. The recommended procedure is to consider the service life to be the sum of the lives of the nonmetallic protective coating, the metallic protective coating, and the basic metal pipe. These three elements can be chosen, mindful of the environmental conditions at the proposed construction site, to ensure a desired design service life. The same relationship can also be used to predict the actual service life for a particular combination of metallic pipe and protective coatings.

11. The California Chart (California Department of Transportation 1972) shown in Figure 1 predicts the time to first perforation (generally in the invert) of galvanized corrugated steel pipe culvert as a function of soil and water pH and resistivity. It is based on a survey of over 7,000 culverts in

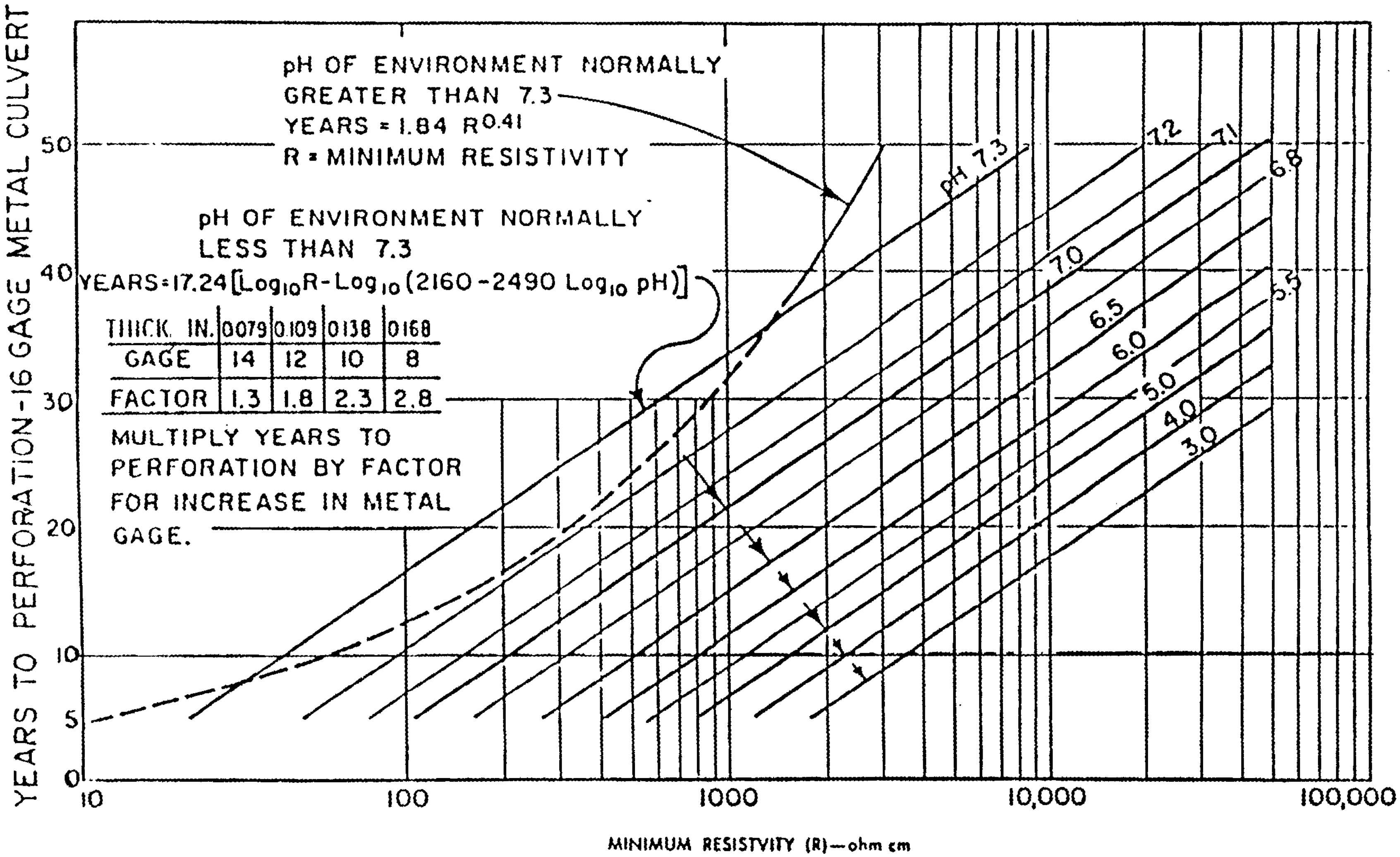

Figure 1. Galvanized steel pipe service life (California Department of Transportation 1972)

California in the 1950's, and is used as a predictor of service life by more states than any other rational method (Task Force 22, 1988). The AISI (1983) Chart (Figure 2) is based on the assumption that culverts can continue to provide service until most of the invert is lost. This point corresponds to a total metal loss approximately twice that corresponding to first perforation. Therefore, the AISI service life was assumed to be double the time to first perforation. However, the assumption of usable life after perforation is only appropriate for gravity flow systems installed in a nonerodible granular bedding. But the Corps of Engineers allows use of silty and clayey sands, which can be highly erodible, and Corps spillways and through-levee structures may operate as pressure systems. In these cases, the time to first perforation is the service life. Further, a study of this issue was recently completed for the California Department of Transportation (CALTRANS) on behalf of the California Corrugated Steel Pipe Association by Mr. George Tupac (1987). He found that the AISI chart is appropriate for the upper 270° of the pipe circumference, but not for the invert. He recommended use of the AISI chart only when the invert is paved. These equations are adjusted for thicker galvanized pipe by multiplying the life, Y, of the 16-gage pipe by a gage factor. These factors are:

Thickness, in.	0.064	0.079	0.109	0.138	0.168
Gage	16.0	14.0	12.0	10.0	8.0
Factor	1.0	1.3	1.8	2.3	2.8

The service life calculated using this method is the average life based on field data. The actual life of individual installations may vary significantly.

12. Aluminum-alloy protective coatings provide better protection for steel pipe than zinc (galvanized) coatings. Long-term field test data (Morris and Bednar 1985) suggest that the aluminum alloy coating (Aluminized Type 2, AASHTO M274) lasts much longer than plain galvanized coatings (zinc, AASHTO M218). The only quantitative data on the actual field performance of Aluminized Type 2 is that contained in the Armco study, so it received close scrutiny, even though it was published in a refereed journal with technical discussions. Supporting information and backup data relating to the

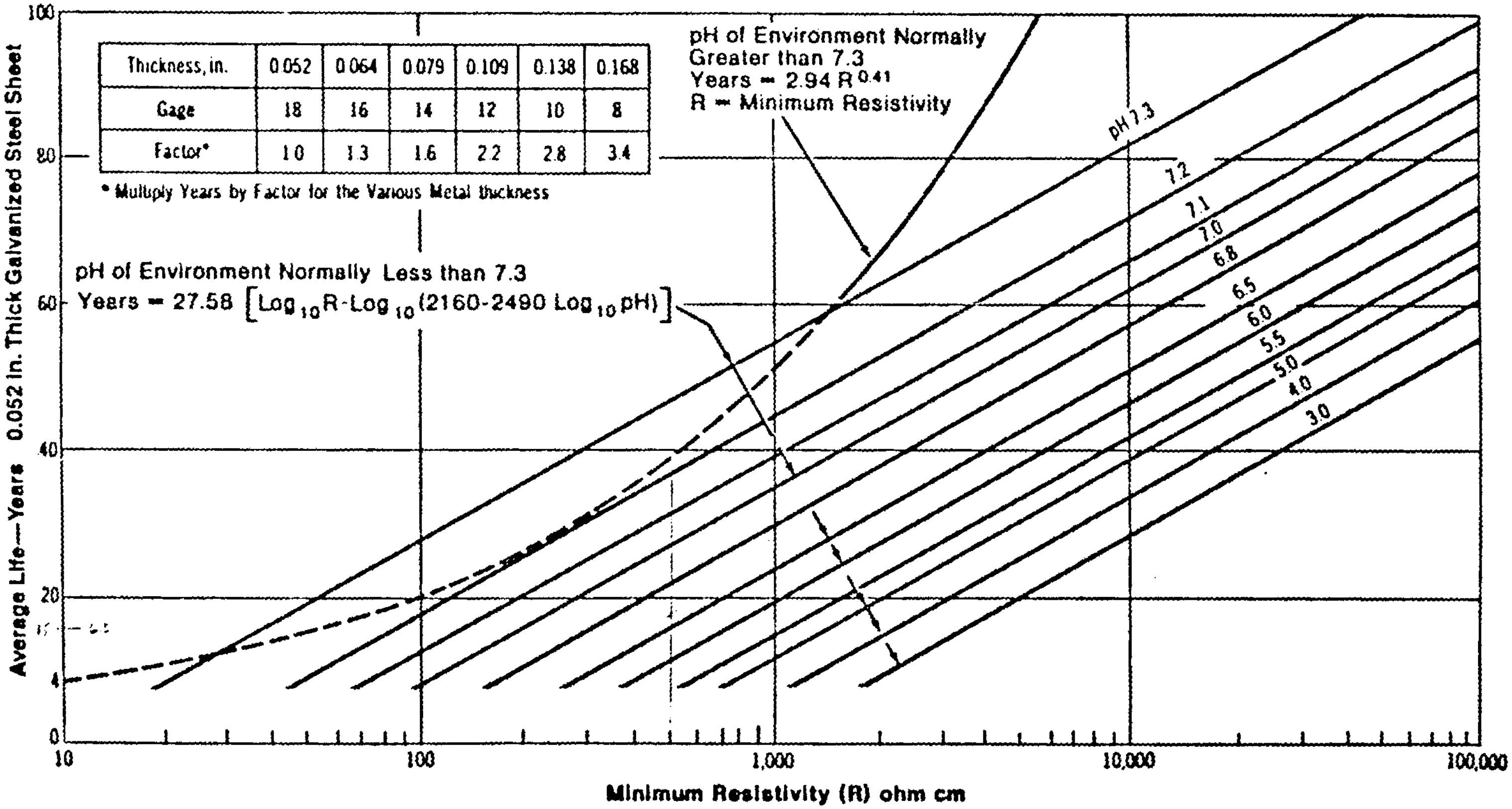

Figure 2. Chart for estimating average life of plain galvanized culverts

performance of Aluminized Type 2, which Armco had used in the preparation of its technical paper or previously prepared for other agencies were also obtained. An independent analysis did show that for 16 gage pipe in the recommended environmental ranges, Aluminized Type 2 lasts two to six times longer than plain galvanized pipe. A comparative factor of two was chosen to be conservative. Thus, the gage factors for aluminum-alloy protective coatings should be:

Thickness, in.	0.064	0.079	0.109	0.138	0.168
Gage	16.0	14.0	12.0	10.0	8.0
Factor	2.0	2.3	2.8	3.3	3.8

However, Aluminized Type 2 should not be used for sanitary or industrial sewers to carry saltwater or acid mine runoff, or where heavy metals are present. The service life of Galvalume (Al-Zn alloy), AASHTO M289 pipe should be calculated as for plain galvanized pipe. Galvalume performs better than galvanized steel in atmospheric exposures (Zoccola et al. 1978), but insufficient published performance data are available to establish this advantage over plain galvanized pipe under the erosive-corrosive conditions typical of most drainage installations.

13. Most of the studies to determine aluminum pit-rate are based on geographical location and not environmental parameters such as pH and resistivity. However, the average pit-rate varies widely over the ranges of pH and resistivity which are recommend for aluminum pipe and which are found throughout the United States. For example, though New York and Maine have established a pit-rate of 0.5 mils per year (mpy), preliminary work by the Louisiana Department of Transportation and Development (Temple and Cumbaa 1986) has placed the pit-rate as high as 2.0 mpy for resistivities below 1,000 ohm-cm. More extensive data are needed to establish a general procedure for estimating aluminum pit-rate.

14. Greater service lives can be achieved by adding the life of an additional nonmetallic coating to the life calculated for the pipe and metallic coating (American Iron and Steel Institute 1983). A synthesis of industry and Government agency policies and recommendations resulted in the following conclusions. Bituminous coating and paving adds about 20 to 25 years to the

average life of the pipe. A bituminous coating alone (AASHTO M190) adds about 8 years for the typical case where water-side corrosion is the dominant influence. Up to 25 years may be added if the effluent is noncorrosive and soil-side corrosion is the critical factor. Note that bituminous coatings are inappropriate for applications where effluents contain petroleum products. Polymer coatings (AASHTO M246), in general, add about 10 years to the average service life. A quality ethylene acrylic film, though, may add up to 20 years to the service life of the base pipe. The most recently published data on the durability of this product cover only 9-1/2 years of exposure. Even using the unpublished reports placing the exposure variously at 13 to 15 years, the use of a life of "30 or more years" requires an extrapolation to more than double the current experience, without benefit of any quantitative data on deterioration rates. An increase in the added life attributable to this promising product is anticipated, as it is proven in field installations. However, currently available data do not support a life much greater than about 20 years.

15. Effects of abrasion were included in the data used to generate the above relationships. Thus, installations with extraordinarily abrasive conditions may experience a shorter service life, but this procedure may conservatively underestimate service life in cases where abrasion is not a factor.

16. Abrasion is a function of velocity and bed load. In the absence of a bed load, abrasion will not be a factor. Also, abrasive materials will not be transported by flows of less than about 5 ft/sec. Therefore, abrasion is not a factor at low velocities without regard to bed load. Abrasion is a factor when abrasive bed loads are present and flow velocities are high enough to transport them. Invert protection should be provided when abrasion is expected to be above the "average" included in the California method. A bed load containing material larger than sand size, with sufficient transport velocity, is likely to produce above average abrasion. Under these particularly abrasive conditions, invert protection should also be considered for aluminum pipe.

Concrete Pipe

17. Concrete pipes may be subject to deterioration from various conditions including freeze-thaw weathering, acid corrosion, sulfate disruption, velocity-abrasion of the concrete, and chloride corrosion of the reinforcing

steel. The reinforcing steel in reinforced concrete pipe may also be subject to corrosion by sulfuric acid resulting from sulfide generation, but this problem only occurs in some sanitary sewers.

18. Precast concrete pipe is generally of high quality and not subject to significant freeze-thaw weathering or chloride corrosion. For cast-in-place structures, the effects of these conditions can be mitigated by ensuring adequate compressive strength (4,000 to 6,000 psi initial strength), limiting the water/cement ratio (to control porosity), and the proper use of admixtures.

19. Acid attack is usually mild when soil and effluent pH's remain between five and seven, and the total acidity is less than 25 mg equivalent to acid per 100 g of soil. No pH related damage has been observed in alkaline environments up to a pH of nine (American Concrete Pipe Association 1981).

20. Sulfate disruption comes from the reaction of sodium, magnesium, or calcium sulfates in the soil and ground water or effluent with the calcium alumina (C_3A) in the concrete, which results in concrete expansion and spalling. This can occur if sulfates are in solution, if there is a differential head between the inside and outside surfaces of the concrete, and if evaporation is taking place on one of the surfaces to concentrate the sulfides. Typically sulfate attack can be a problem when the sulfate content exceeds 1,000 parts per million. Under these conditions, use of Type II or Type V cements will impede deterioration. The Bureau of Reclamation (US Department of Labor 1975) guidelines shown in Table 1 can be used to control the effects of sulfate attack. Other strategies for reducing the deleterious effects of sulfates from the Bureau of Reclamation include reducing the C_3A content of the concrete, steam curing, decreasing the absorption factor, and increasing the cement content. The California Department of Transportation considers a seven sack mix using Type II cement to be equivalent to a mix using the minimum allowable amount of Type V cement (Transportation Research Board 1978).

21. Abrasion is a function of velocity and bed load. Abrasion is not a factor when velocities are less than 15 ft/sec. Some additional protection is required for velocities between 15 and 40 ft/sec if a bed load is present. This protection may be in the form of increased cement content such as an eight sack mix, increased concrete cover over reinforcing (typically 1-1/2 in. and/or harder aggregates) or both. In the absence of bed loads, velocities of

Table 1

Attack on Concrete by Soils and Waters Containing Various Sulfate Concentrations

Relative Degree of Sulfate Attack	Percent Water-Soluble Sulfate (as SO_4) in Soil Samples	Parts Per Million Sulfate (as SO_4) in Water Samples
Negligible	0.00 to 0.10	0 to 150
Positive*	0.10 to 0.20	150 to 1,500
Severe**	0.20 to 2.00	1,500 to 10,000
Very severe†	2.00 or more	10,000 or more

* Use Type II cement.

** Use Type V cement or approved portland-pozzolan cement providing comparable sulfate resistance is used in concrete.

† Use Type V cement plus approved pozzolan which has been determined by tests to improve sulfate resistance when used in concrete with Type V cement.

up to 40 ft/sec can be accommodated. Cavitation may produce serious damage if velocities exceed 40 ft/sec, because of the geometry of reinforced concrete pipe joints.

22. Hydrogen sulfide gas may be generated in sanitary sewers. However, the buildup of sulfide gas can generally be prevented by maintaining a minimum flow velocity of 2 ft/sec. This velocity also provides for efficient solids transport. Where this control strategy is not practical, a more comprehensive design for sulfide control is required. Sulfide generation, the investigation and prediction of sulfide levels, and the rate of sulfide corrosion are discussed in detail in the "Concrete Pipe Handbook" (American Concrete Pipe Association 1981) and the "Design Manual for Sulfide and Corrosion Prediction and Control" (American Concrete Pipe Association 1984).

23. Within the environmental constraints outlined above, the service life of concrete pipes varies significantly. The most extensive survey of state guidelines on service life was performed by the New York State Department of Transportation (Renfro and Pyskadlo 1980). Ring (1984) reported that the assumed useful lives obtained from this survey ranged from 20 to 75 years but had an average of 56.5 years.

24. There are relatively few detailed studies on concrete pipe durability. Most are state department of transportation reports. Since each report is for a particular state and its environmental conditions, variables which affect pipe durability but which do not vary significantly across the state were frequently not included.

25. The most complete data are those collected by the Ohio Department of Transportation (Meachan, Hurd, and Shisler 1982). During the period 1972 to 1975, 545 concrete pipe culverts were inspected. All of the culverts were located in Ohio with the most acidic sites concentrated in the coal-field region of southeastern Ohio. Fourteen of the cases had clay liner plates, and slope or sediment depth measurements were omitted at 132. This leaves 399 complete observations. The ranges of the variables are shown in Table 2.

Table 2

Variables Used in the Ohio Studies

Variable Name	Description	Units	Range
Rate	Pipe condition rating (Table 3)	--	1 - 5
Age	Pipe age	years	1 - 45
Rise	Pipe vertical diameter	inches	24 - 108
Flow	Water flow velocity index (1=rapid, 2=moderate, 3=slow, 4=negligible, 5=no flow)	--	1 - 5
Sed	Sediment depth in invert	inches	0 - 60
Slope	Pipe slope	percent	0.01 - 58
pH	pH of water inside pipe	--	2.4 - 9.0

26. Hurd (1984) developed a service life equation based on a subset of these data. He used only data from pipes with diameters greater than 42 in. to improve the accuracy of the condition rating and increase the probability of dry weather flow. Also, he used only the data with a pH of less than 7.0 on the assumption that acid attack is the principal deterioration mechanism, and since there can be no acid attack at pH's of 7.0 and greater, these data need not be included. These selection criteria resulted in a data set consisting of 45 cases. A multiplicative (nonlinear) regression equation for

condition rating (as the dependent variable) was fitted to these data. That equation was then solved for pipe age with the condition rating fixed at a terminal value (4.5) to produce the service life equation.

27. Later, Hurd (1985) revised and expanded his acid-site data set. He included more acid-site culverts from the 1972 survey and improved the accuracy of the data. Pipes were reinspected and rerated according to a less subjective rating criteria, and the new data were checked against the old data to detect changed conditions, recording errors, or other anomalies. The 1984 survey resulted in an acid-site data set with 59 cases.

28. Hurd's regression equation for the condition rating of these 59 sites is:

$$\text{rating} = \frac{[6.501\ (\text{age})^{0.55}\ (\text{rise})^{1.079}\ (\text{slope})^{0.233}\ (\text{pH})^{-3.079}]}{[1 - (\text{sed}/\text{rise})]^{1.465}} \qquad (1)$$

where

rating = pipe condition rating from 0 to 100 with 0 being as manufactured and 95 being the end of useful life

age = pipe age at time of inspection, years

rise = pipe vertical diameter, in.

slope = invert slope, percent

pH = water pH

sed = sediment depth in pipe invert, in.

29. The pipe rating for a site with a pH of 7 or above is assumed to be less than or equal to the rating given by Equation 1 using a pH of 7.0. Equation 1 has an r^2 of 0.6.

30. Hadipriono (1986) also used the Ohio data to model pipe rating. He fitted a linear, additive regression equation for pipe rating to the complete 399 case data set. He contended that factors in pipe deterioration such as weathering, velocity abrasion, and sulfate disruption contributed to pipe deterioration regardless of pH. Thus, data from the entire pH range were included. He also included pipe sizes less than 42 in. in order to take advantage of the information contributed by these cases. Hadipriono grouped the pipes by rise (diameter) so that his model actually consisted of four equations:

$$\text{rate} = 6.1040 + 0.03082\ (\text{age}) - 0.073\ (\text{flow}) - 0.0139\ (\text{sed}) - 4.9425\ (\log \text{pH}) + 0.1276\sqrt{\text{slope}} \quad \text{for rise} \leq 42 \text{ in.} \tag{2}$$

$$\text{rate} = 6.2472 + 0.03082\ (\text{age}) - 0.073\ (\text{flow}) - 0.0139\ (\text{sed}) - 4.9425\ (\log \text{pH}) + 0.1276\sqrt{\text{slope}} \quad \text{for } 42 \text{ in.} < \text{rise} \leq 48 \text{ in.} \tag{3}$$

$$\text{rate} = 6.1946 + 0.03082\ (\text{age}) - 0.073\ (\text{flow}) - 0.0139\ (\text{sed}) - 4.9425\ (\log \text{pH}) + 0.1276\sqrt{\text{slope}} \quad \text{for } 48 \text{ in.} < \text{rise} \leq 60 \text{ in.} \tag{4}$$

$$\text{rate} = 6.4770 + 0.03082\ (\text{age}) - 0.073\ (\text{flow}) - 0.0139\ (\text{sed}) - 4.9425\ (\log \text{pH}) + 0.1276\sqrt{\text{slope}} \quad \text{for rise} > 60 \text{ in.} \tag{5}$$

where

rate = pipe condition rating from 1 to 5 with 1 being excellent and 4.5 being the end of useful life

flow = water flow velocity rating from 1 to 5 with 1 = rapid, 2 = moderate, 3 = slow, 4 = negligible, and 5 = no flow, and the other variables are as previously defined

Equations 2 to 5 can be combined for simplicity (with a new regression analysis):

$$\text{rate} = 5.7478 + 0.0304\ (\text{age}) - 0.0752\ (\text{flow}) - 0.0134\ (\text{sed}) - 4.8920\ (\log \text{pH}) + 0.1264\sqrt{\text{slope}} + 0.0085\ (\text{rise}) \tag{6}$$

The r^2 remains about 0.4, and the relationship of rate to rise becomes a smooth monotonic function.

31. Equations 1 and 6 and their respective r^2 values cannot be compared directly because of the differences in condition rating scale, flow velocity rating (0.1 to 3 for the acid-site data), and size of the data set. A direct comparison requires interchanging data sets. Also, since the rating scales are different, the regression analysis must be repeated to establish new coefficients.

32. Using Hurd's 59 cases instead of 399, the additive, linear regression equation for pipe rating is:

$$\begin{aligned} \text{rating} = {} & 134.1330 + 0.4133\,(\text{age}) + 4.0657\,(\text{vel}) - 1.3490\,(\text{sed}) \\ & - 188.4258\,(\log \text{pH}) + 5.3113\sqrt{\text{slope}} + 0.1947\,(\text{rise}) \end{aligned} \tag{7}$$

where

vel = water flow velocity rating from 0.1 to 3, with 0.1 = nil, 1 = slow, 2 = moderate, and 3 = rapid, and the other variables are as previously defined

With the smaller data set, the r^2 increases from 0.4 to 0.7. The r^2 for the multiplicative regression equation drops from 0.6 to 0.3 when the larger data set is used. The r^2 values are compared below:

	Equation	
Cases	Hadipriono (1986)	Hurd (1985)
59	0.7	0.6
399	0.4	0.3

This comparison suggests that the form of Hadipriono's equation (additive, linear) more closely models the phenomenon of pipe deterioration than Hurd's equation (multiplicative, nonlinear) for either set of data. However, neither can be described as a "good" model with these low r^2 values.

33. Most empirical performance or service life relationships are based on formulas fitted to data from specimens at some terminal or failed condition. The age used for these equations is the service life. When deterioration models such as Equations 1 and 6 are used to predict service life, another level of complexity is added. The age in these equations is the age of the specimen at the time of inspection when a rating was assigned. These equations are used to predict service life by solving for age and setting the condition rating to some terminal value. There may be very little data with rating values at or near terminal. Thus, the service life prediction presumes a relationship for deterioration with age in addition to relationships between the other independent variables and the service life. The form of the relationship between age and rating is critical because it is used to forecast a

service life based on data from specimens which have not yet failed or performed for their full service life. This relationship may or may not be linear. In fact, Hurd contends that it is highly nonlinear. Hurd's equation (Equation 1) can account for nonlinearity, but Hadipriono's (Equation 6) cannot. However, it is important to note that the exponents in multiplicative equations such as Equation 1 actually serve two purposes. They serve to define the shape of the relationship, which is nonlinear when the exponent has a value other than one. They also serve as scaling or weighting factors to reflect the relative influence of the independent variables. The exponent of an important independent variable must be larger than the exponent of a lesser variable to make the correct variable dominate the behavior of the dependent variable if the relative magnitudes of the independent variables and the forms or shapes of their relationships to the dependent variable are similar. A regression analysis does not distinguish between adjustments to the exponent for weighting and adjustments for improving the shape of the relationship. Producing Equation 1 by regression analysis does not prove that the relationship between age and rating is highly nonlinear. For example, if age had been expressed in months or days instead of years, the exponent on age would have been much smaller. Yet, the shape of the true relationship of rating with time must be the same.

34. Solving Equation 6 for age and setting rate to a terminal value of 4.5 gives the following predicted service life:

$$\begin{aligned}\text{service life} = {} & 41.05 + 2.47\ (\text{flow}) + 0.44\ (\text{sed}) \\ & + 160.92\ (\log \text{pH}) - 4.16\sqrt{\text{slope}} - 0.28\ (\text{rise})\end{aligned} \tag{8}$$

35. The depth of sediment in the invert (sed) is difficult to predict and is generally undesirable with regard to culvert hydraulics. Therefore, a sediment depth of zero should be used for design. The flow velocity is also difficult to predict and is usually highly variable. An average value is reasonable for initial design and economic analysis. Using the average flow velocity for the complete data set (3.1604) and zero sediment depth, Equation 8 reduces to:

$$\text{service life} = -33.23 + 160.92\,(\log \text{pH}) - 4.16\sqrt{\text{slope}} - 0.28\,(\text{rise}) \tag{9}$$

36. By comparison, solving Equation 1 for age at a terminal rating of 95 (comparable to 4.5) and 0 sediment depth gives

$$\text{service life} = 123.5\,(\text{pH})^{5.55}\,(\text{rise})^{-1.94}\,(\text{slope})^{-0.42} \tag{10}$$

37. Equations 9 and 10 give very similar service lives for values of pH between 2 and 4 but differ dramatically at higher values of pH. This departure results largely from the large value of the exponent (5.55) on pH in Equation 10.

38. Both Equations 9 and 10 are based on culverts generally not at the end of their useful lives. However, the times given by Equations 9 and 10 should be consistent with the ages of culverts rated 4.5 to 5 in the complete data set and 95 to 100 in the acid-site data set if they accurately predict service life.

39. There are 20 such data points between the two data sets having rises from 30 to 108 in. and slopes from 0.02 to 18 percent. The average rise is 57 in. and the average slope is 2.8 percent. Figure 3 shows Equations 9 and 10 plotted using these average values for rise and slope along with the age and pH of culverts rated 4.5 to 5 or 95 to 100.

40. The two culverts rated 5 after only 3 and 4 years appear to be outliers. Their poor performance may be the result of a substandard pipe or other unusual deleterious factor not observed during the inspections. The remaining 18 points appear to plot in an orderly and logical manner, although there is considerable scatter as noted earlier.

41. Equations 9 and 10 both fit the data reasonably well in the pH range of 2 to 4. However, both appear to overestimate service life for values of pH greater than 4. This is caused by the forecasting or prediction of service life based on the ages of culverts still in good condition. A review of the data prepared reveals that there are many culverts still in good condition at ages near that of the data shown in Figure 3. This observation suggests that factors not evaluated are significantly influencing culvert performance.

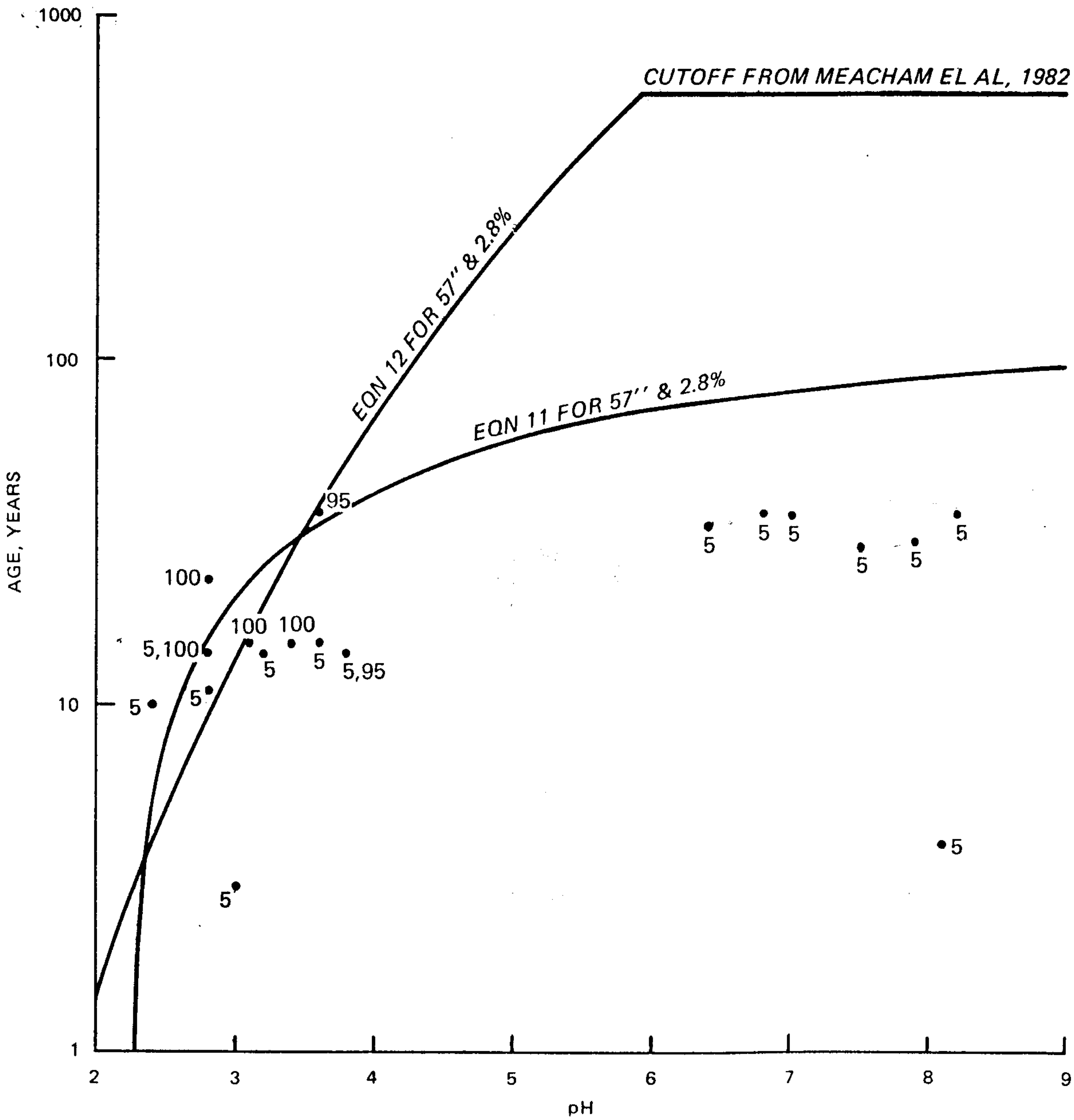

Figure 3. pH versus age for failed pipes with assigned rating

One possibility is variation in pipe quality. If the data in Figure 3 represents relatively lower quality pipe, then typical or average quality pipes would last longer. Thus, a longer life than that indicated by the failure data would be expected for average quality pipes. Equation 9 is a reasonable candidate for describing average service life under this philosophy. Its fit to the data, based on its r^2 value and Figure 3, leaves much to be desired, but it is a point of departure for future improvements. It also appears to be more appropriate than Equation 10. Equation 10 is not based on performance data for values of pH greater than 6.9; yet it implies a service life for these environments. Further, this implied service life in alkaline environments is very large. The 600 year limit placed on Equation 10 by Hurd (1985) is more than an order of magnitude larger than the age of any of the culverts used in its determination.

42. Both Equations 10 and 11 suggest that the service life will be in excess of 50 years as long as the pH exceeds 4. As the pH becomes less acidic, the service life can be expected to be much longer. The maximum service life occurring in an alkaline environment cannot be determined with confidence, but it should exceed 100 years, and may be as high as 600 years.

Plastic Pipe

43. Plastics are combustible, and many are subject to attack by ultraviolet light if not buried or otherwise protected. The chemicals that are known to deteriorate plastic pipes are not normally found in culvert and storm drainage effluents.

44. Plastic pipes can be broadly classified as either thermoplastic or thermosetting. These designations are derived from the kinds of polymers that are used in their manufacture. Thermosetting plastics are used in reinforced plastic mortar pipe (RPMP) and reinforced thermosetting resin pipe (RTRP). RTRP is usually glass reinforced plastic (GRP) or fiber reinforced plastic (FRP). The principal thermoplastics are polyvinyl chloride (PVC), polyethylene (PE), and acrylonitrile butadiene-styrene (ABS). Others are chlorinated polyvinyl chloride (CPVC), polybutylene (PB) and polypropylene (PP). PVC and high density polyethylene (HDPE) are the principal materials used in drainage structures. Additional information can be found in the resources listed in the bibliography.

45. The most significant characteristic of these pipe materials is that they exhibit a viseoelastic response to thermo-mechanical loading (Chaturvedi 1986). The effective, long-term elastic modulus is lower than the short-term modulus due to creep in the loaded material as a function of load and temperature. This property must be explicitly considered in the structural design to ensure long-term service life. State of the art design procedures (Schluter 1985, Chambers and McGrath 1986) use a long-term elastic modulus less than the initial modulus to account for long term pipe behavior. Variations on these procedures have been adopted by various standardization organizations.

46. One of these procedures is the proposed American Association of State Highway and Transportation Officials (AASHTO) plastic pipe design procedure "Section 18: Soil Thermoplastic Pipe Interaction Systems" (Appendix A). The structural design of thermoplastic pipes larger than 6 in. nominal size should follow this procedure. The structural design of smaller thermoplastic pipe may also use this method. The use of effective modulus to define the ultimate deformation response is based on constant and continuous loading. In practice, these two conditions are seldom met throughout the anticipated life of the pipe, and therefore, the concept of effective, creep modulus has some inherent safety factor, the magnitude of which depends upon actual conditions. Hence, life under loads based on 50-year modulus values should be significantly greater than 50 years and for longer still, under even lighter loads. Table 3 is an example of a cover table based on this procedure. It gives the maximum and minimum cover requirements for one particular corrugated polyethylene pipe subject to H20 live loads using 50-year modulus values. Other pipe constructions, with both corrugated or solid walls, are commercially available for greater maximum cover heights.

47. Pipes made from thermosetting plastics (RTRP, RPMP) should be designed and installed in accordance with American Society for Testing and Materials (ASTM) D 3839-79 (ASTM 1979).

48. For thermoplastic pipes (PE, PVC, CPVC, PB, CAB), 6 in. nominal size and smaller, ASTM D 2321-83 should be followed for design and installation.

49. Smooth wall, high density polyethylene pipe has demonstrated abrasion resistance 3 to 5 times greater than mild steel, documented in ETL 1110-3-332 (Headquarters, Department of the Army 1986).

Table 3

Pipe Cover Requirements for Corrugated Polyethylene Pipe Subject to H20 Live Loads

Nominal Diameter in.	Minimum Cover ft	Maximum Cover ft
12	1.0	9.6
15	1.0	9.7
18	1.0	10.0
24	1.0	10.3

Notes:

a. The suggested maximum heights of cover shown in the table are calculated on the basis of the proposed AASHTO standard specifications for highway bridges, Section 18: Soil-Thermoplastic Pipe Interaction Systems using service load design and assuming a soil density of 120 pcf.

b. Cover depths are measured from the top of the pipe to the top of the ground surface.

c. Regardless of minimum cover requirements, the distance from the top of the pipe to the bottom of the slab of rigid pavements must exceed the values given in the following tabulation (extracted from TM 5-820-3) (Headquarters, Department of the Army 1981) to prevent cracking of the slab.

Pipe Size in.	Gear Load: Less Than 100 Kips, ft	Gear Load: 100 Kips or Greater, ft
6-60	0.5	1.0
66-120	1.0	1.5

Clay Pipe

50. Vitrified clay is perhaps the least corrodible of the common pipe materials. It is subject to corrosive attack only from hydroflouric acid and concentrated caustics. It is also very resistant to abrasion (Bortz 1985). As a result, vitrified clay is extremely durable in terms of deterioration from corrosive or abrasive service environments.

51. The National Clay Pipe Institute (1982) has compiled a list (Table 4) of over 50 clay pipe systems which are still functioning after up to 170 years and which are used to support a 150-year service life. However,

Table 4

Old Clay Pipe Installations Still in Service*

	City	Date Installed		City	Date Installed
1.	Washington, DC.	1815	27.	Baltimore, Md.	1875
2.	Philadelphia, Pa.	1829	28.	Portland, Maine	1875
3.	Boston, Mass.	1829	29.	San Francisco, Calif.	1876
4.	Sydney, N.S. Wales	1832	30.	Jacksonville, Fla.	1876
5.	Manchester, England	1845	31.	Albany, Ga.	1876
6.	Liverpool, England	1846	32.	St. Joseph, Mo.	1876
7.	London, England	1848	33.	Davenport, Iowa	1877
8.	Clinton, Iowa	1850	34.	Kansas City, Mo.	1877
9.	Edinburgh, Scotland	1850	35.	New Bedford, Mass.	1877
10.	Rigby, England	1851	36.	Bucyrus, Ohio	1877
11.	Croydon, England	1851	37.	Omaha, Nebr.	1878
12.	Darlington, England	1852	38.	Camden, N.J.	1879
13.	Chicago, Ill.	1856	39.	Memphis, Tenn.	1879
14.	Cleveland, Ohio	1861	40.	Parkersburg, W. Va.	1879
15.	New York, N.Y.	1866	41.	Providence, R.I.	1879
16.	Erie, Pa.	1868	42.	Nashville, Tenn.	1879
17.	Grand Rapids, Mich.	1869	43.	Rome, Ga.	1880
18.	St. Louis, Mo.	1869	44.	Rockford, Ill.	1880
19.	Hartford, Conn.	1870	45.	Terre Haute, Ind.	1880
20.	Indianapolis, Ind.	1872	46.	Sioux City, Iowa	1880
21.	Los Angeles, Calif.	1873	47.	Red Wing, Minn.	1880
22.	New Haven, Conn.	1873	48.	Reno, Nev.	1880
23.	St. Paul, Minn.	1873	49.	Fargo, N. Dak.	1880
24.	Portland, Oreg.	1873	50.	Dallas, Tex.	1880
25.	Raleigh, N.C.	1873	51.	Denver, Colo.	1880
26.	Lawrence, Kans.	1874			

* From National Clay Pipe Institute 1982.

most of the referenced systems are just over 100 years old. Thus, and particularly in light of the uncertainty in long-term (over 100 years) land use, it is appropriate to limit the design service life of vitrified clay pipe to 100 years.

Summary

52. A 50-year service life can be used for most types of drainage structures. Limits on pH and resistivity can be used to ensure that metal pipes will perform satisfactorily for this period. Also, the California method, along with the added life afforded by protective coatings, can be used to estimate the service life of a corrugated steel pipe or develop a combination of pipe and coating to last 50 years in a particular environment.

53. Limits on pH and sulfides can be used to ensure the satisfactory performance of concrete pipes. As the pH increases from 4 to 9, reinforced concrete pipe life increases from about 50 years to over 100 years, depending on pipe diameter and slope. As with metal pipe, there is considerable variability in actual service life, and the available data cannot be used to confidently estimate service life.

54. Plastic pipe should provide much more than 50 years of service as long as it is not exposed to ultraviolet light and the structural design is based on the long term creep behavior of the plastic. The proposed AASHTO design procedure is one such procedure and may be used pending its adoption.

55. Clay pipe is perhaps the most inert of the common pipe materials in terms of corrosion, and it is very resistant to abrasion. A 100-year service life may be assumed for most clay pipe installations.

PART III: LIFE CYCLE COST METHODOLOGY

General

56. The first step in the analysis of design alternatives is to develop a preliminary list of all possible alternatives. This list is then reduced to a group of feasible alternatives by applying the constraints of the particular project such as availability of materials or equipment, site conditions such as abrasive bed load, or requirements to accommodate large flows or livestock. The minimum functional requirements must be met. The final design is chosen from this group based on LCC.

57. The LCC is the total, overall estimated cost for a particular design alternative. Direct and indirect initial costs plus periodic or continuing costs for operation and maintenance are included. The methods described in TM 5-802-1 (Headquarters, Department of the Army 1986) and mentioned below account for the time value of money and reflect the concepts and procedures used in many economics texts (Theusen, Fabrycky, and Theusen 1971).

58. Costs incurred over time may be expressed in terms of either constant dollars or current dollars. Constant dollars are costs (or savings) stated at price levels in effect at some given time, usually the particular time that the analysis is conducted. Current dollars are costs or savings stated at price levels in effect whenever the costs or savings are incurred. Comparison of drainage structure alternatives should be based on constant dollars for all costs including present and future costs and for salvage or retention/residual values.

59. The LCC is expressed either in terms of present worth (PW) or equivalent uniform annual cost (EUAC). PW is the primary measure of LCC. It is the amount of money required now to fund the project for the entire analysis period. The EUAC is the amount of money required for each year of the analysis period to fund all project costs.

60. The same analysis period must be used to compare alternatives using PW's. PW's can be converted to EUAC using a uniform series capital recovery factor. In this case, PW and EUAC are just two ways of expressing the same costs. EUAC can also be calculated from the individual costs for each alternative.

Analysis Period

61. Economic studies consider projects which have a service life, an economic life, and an analysis period. The service life is the total useful life of the project or time to replacement or rehabilitation. The economic life is the time during which a project is economically profitable or provides the required service at a lower cost than another facility. For drainage structures, the economic life is usually the same as the service life. The analysis period is the comparison period over which costs are counted in determining the PW or EUAC of an alternative.

62. Guidance for selecting the analysis period is given in AR 11-28 (Headquarters, Department of the Army 1975) as shown below:

> The alternative with the longest economic life may determine the end of the comparison period. However, the decision maker or analyst may shorten this period consistent with the objectives and assumptions of the analysis. Whether the longest or shortest life is used as a basis, adjustment for unequal life is required. If the shortest life is used the residual values of the alternatives with longer lives must be recognized in the cost computation for those alternatives. Should the longest life be used to establish the time period of the analysis, the cost of extending the benefit producing years of those alternatives with a shorter life must be recognized. Care should be exercised to ensure that the costs for each alternative for the entire period of comparison are presented to the decision maker. Another alternative would be the use of uniform annual cost methods as a means of comparison [5, p. 2-5].

63. TM 5-802-1 further limits the analysis period to the economic life or 25 years, whichever is less. The 25-year limit is based on the projected economic life of the complete facility encompassing the drainage structure, which is usually around 25 years for general planning purposes. However, infrastructure such as drainage facilities may realistically be expected to provide economical service, in its original mission, well beyond 25 years. A review of the service lives used by various state and federal Government agencies and industry (Renfro and Pyskadlo 1980, Summerson 1984) reveals that most agencies expect culverts to provide service longer than 25 years with a 50-year life used most frequently. This period strikes a balance between the

intangible and/or indirect costs associated with replacement or rehabilitation, and the unpredictability of long-term land use. Based on the service life guidelines for metal, concrete, plastic, and clay pipes (Part II), a 50-year analysis period is justifiable and should be used, subject to the approval as described in TM 5-802-1 (Headquarters, Department of the Army 1986) of HQUSACE (CEEC-EG) for Army projects and HQUSAF (LEEEC) for Air Force projects.

Costs

64. The initial and recurring costs considered in an economic analysis are sometimes categorized as agency costs, user costs, and nonuser costs (Hass and Hudson 1978). Agency costs include initial capital costs of construction, future capital costs of rehabilitation or replacement, maintenance and/or operational costs during the analysis period, salvage or retention/residual value (a negative cost) at the end of the analysis period, and engineering and administrative costs. User costs are usually included in the cost of the facility being drained by the drainage structure. Those costs include travel time, vehicle operating costs, accident costs, and inconvenience (as when a detour is required). Nonuser costs result from the impact of the facility on those not actually using the facility such as the cost of flood damage occurring downstream of the drainage structure.

65. Economic analyses frequently include only the initial and future capital costs, maintenance and operation costs, and salvage or retention/residual value. For drainage structures, the other costs are likely to be similar for all alternatives. Thus, little error is introduced by omitting them from the computations. One exception is the user cost associated with replacement of a structure during the analysis period. Replacement of structures under high-volume facilities may cause expensive delays and detour costs, as well as reconstruction costs well in excess of the marginal cost associated with the initial installation of the structure.

66. Initial capital costs for drainage structures can generally be estimated from local data, usually obtainable from local vendors. Future capital costs except as noted, can be estimated from current costs, adjusted as necessary for the time expected before future construction. As a supplement, or if local data are not available, costs can be estimated using the

procedures, rates, and adjustment factors given in AR 415-17 (Headquarters, Department of the Army 1980), Engineering News Record's Building and Construction Cost Index Histories (Engineering News Record 1987), the Highway Maintenance and Operation Cost Trend Index (Federal Highway Administration 1987), and the Price Trends for Federal-Aid Highway Construction (Federal Highway Administration 1987). A description of these resources and their use are included in Kohn, Epps, and Rosser (1987).

67. Maintenance and operations costs are best determined from local experience with similar projects. Maintenance and operations costs are highly dependent upon both local conditions and the particular maintaining agency.

68. The salvage or retention/residual value of a drainage structure is its residual value at the end of the analysis period. If the end of the analysis period coincides with the end of the service life of the alternative, then the salvage value of that alternative can probably be taken as zero. When the service life is expected to exceed the length of the analysis period, the retention/residual value must be included, generally as a future income or negative cost.

Discount Rate

69. The time value of money is expressed by the discount rate. The discount rate is the amount that the value of money in the future is reduced or discounted to reflect its present value. It can also be viewed as the minimum real or net rate of return, after inflation, to be achieved by public sector investments. Congress has stipulated that diverting investment capital from the private sector (by taxation) can only be justified when that capital is used on public-sector projects having a real rate of return at least as high as that achievable in the private sector. Through OMB Circular A-94 (Office of Management and Budget 1972), this rate has been set at 10 percent.

Computing Present Worth

70. The basic method for computing the PW of a given alternative is described in detail in TM 5-802-1 (Headquarters, Department of the Army 1986) and summarized here:

a. One-time costs.

(1) Step 1: Estimate the amount of the one-time cost as of the base date (date of the study).

(2) Step 2: Escalate this cost to the time at which it is actually to be incurred using the differential (from inflation) escalation rate e.

(3) Step 3: Discount the escalated future one-time cost to PW (on the base date) using the discount rate d (currently 10 percent).

b. Recurring costs.

(1) Step 1: Estimate the amount A_o of the annually recurring cost as of the base date, and determine the number of costs, k, in the series (e.g. over the analysis period).

(2) Step 2: Escalate A_o to A_i at the time at which the first cost in the series is to be incurred using the escalation rate e.

(3) Step 3: Determine, for the date on which A_i is incurred, the single cost that is equivalent to a series of k uniformly escalating annual costs where the amount of the first cost is A_i and the escalation rate is e.

(4) Step 4: Discount the single equivalent cost, from the time the first annual cost is to be incurred to a PW on the base date using the discount rate d.

Formulas, tables, and sample calculations are provided in Technical Manual TM 5-802-1.

Decision Criteria

71. Uncertainty in LCC and LCCA is discussed in TM 5-802-1:

> The input data for an LCCA are based on estimates rather than known quantities and are, therefore, uncertain. They may be uncertain as to the scope or quantity of things (e.g., pounds of steel, manhours of labor), the unit costs of things in the marketplace at the time the costs will actually be incurred, and the

timing of cost (e.g., when a floor covering will require replacement). The effects of uncertainties on the results of an LCCA can be quite significant. They may distort the results of the analysis or dominate them so that one alternative may appear to be lowest in net LCC under one set of reasonable assumptions and highest in net LCC under another equally reasonable set of assumptions. For these reasons, the need for uncertainty assessment will be considered as part of every LCCA.

a. Specific requirements. The decision as to whether or not an uncertainty assessment is required for any particular LCCA will depend on a number of factors and so must be made on a case-by-case basis. Among these factors are whether or not the LCCA results appear to be clear-cut, whether or not the relative economic rankings of the (apparently) top-ranked alternative and its nearest competitors could be affected by the results of the assessment, whether or not the LCCA results have to be approved by higher Command authority prior to implementation, and whether or not the LCCA results are likely to be controversial (as are deviations from criteria, changes from common practice, rejections of special user preferences, and significantly greater initial cost requirements that result in only marginal LCC savings). In general, an uncertainty assessment need not be performed if either of the following conditions applies.

(1) The relative economic rankings of the (apparently) top-ranked alternative and its nearest competitors cannot be affected by the results of the assessment.

(2) The LCCA results appear to be clear-cut, either clearly conclusive or clearly inconclusive, in advance.

In addition, even if the LCCA results appear not to be clear-cut (i.e., not clearly conclusive and not clearly inconclusive) (especially the latter), an uncertainty assessment is not considered necessary provided the design decision is a routine one (i.e., one which may be implemented locally without the need for higher-authority approval) and is one that is unlikely to be controversial when implemented.

b. Approaches. Of the two leading approaches to uncertainty assessment, the probabilistic approach is more direct and the generally applicable for MCP designs, and it should be used whenever appropriate. Since the rigorous probabilistic approach is too complex for routine use, reasonable approximations to that approach are preferred for MCP design applications. The other leading approach to uncertainty assessment, the sensitivity approach, may be used in any situation in which the approach is valid; however, in all cases in which the probabilistic approach and the sensitivity approach are both valid, the probabilistic approach is preferred. In those situations where neither the probabilistic approach nor the sensitivity approach can be considered to be valid, uncertainty assessment may be accomplished by means of any common-sense heuristic approach--preferably one based on either the probabilistic or the sensitivity approach, or on some combination of the two (para 2-2. b. (9)).

72. In the case of a tie between any of the alternatives, the relative ranking can be determined using the following guidance, also from TM 5-802-1:

> If any alternatives are determined to have comparable net LCC's either because their calculated net LCC's are essentially equal or because the uncertainties associated with the analysis are found to be sufficiently large to render apparent net LCC differences inconclusive, then their relative rankings will be based on a combination of energy conservation and initial procurement cost considerations, as outlined below. For those situations in which the LCCA results appear not to be clear cut, the criteria for judging whether apparent net LCC differences are conclusive or inconclusive and, hence, whether the LCCA results are conclusive or inconclusive are as follows:
>
> a. A positive net LCC difference between two alternatives is conclusive if it can be shown that the probability of that difference exceeding zero is no less than 0.60.
>
> b. A positive net LCC difference between two alternatives is inconclusive if it can be shown that the probability of that difference exceeding zero is no greater than 0.55. Finally, in the absence of net LCC determinations either because an LCCA has not been conducted or because one has been conducted but not in strict accordance with the criteria contained herein (e.g., it was not based on the best information available at the time), design alternatives will be given economic rankings based solely on initial procurement cost considerations.
>
> c. Tie breaking. If two design alternatives have comparable net LCC's, and it can be demonstrated with a high degree of confidence that one of these alternatives satisfies any of the following conditions, then that alternative will be assigned the higher relative ranking:
>
> (1) It will be less expensive in terms of initial procurement costs and will consume no more fuel/energy per year.
>
> (2) It will consume less fuel/energy per year and will be no more expensive in terms of initial procurement costs.
>
> (3) It will consume at least 15 percent less fuel/energy per year and will not be more than 15 percent more expensive in terms of initial procurement costs.

(4) It will be at least 15 percent less expensive in terms of initial procurement costs and will consume no more than 15 percent more fuel/energy per year.

When the two alternatives are of different fuel/energy types, quantities of fuel or energy consumed annually will be determined in Btu equivalents, measured at the source, in accordance with standard practice within the Department of Defense for measuring energy savings. If none of these conditions are satisfied, then the two alternatives will be assigned the same ranking. In those cases when two or more of the alternatives considered for any design feature are tied for the highest ranking, selection will be based on the designer's judgement as to which of the alternatives tied for the top ranking represents the best overall choice in terms of initial cost, energy consumption, and LCC for the application at hand (para 2-2. c.).

Example

73. Suppose a drainage structure is being selected for construction 2 years after the analysis base date (date of study). The soil/water pH is 6.0, the minimum soil/water resistivity is 6,000 ohm-cm, and a nonabrasive flow of 6 ft/sec is expected. The facility being drained is a low volume road with shallow pipe cover, so replacement costs are similar to initial construction costs, and no significant user costs are expected from delays or detours. The materials to be considered are reinforced concrete (RCP), plain galvanized (CSP), asphalt coated and paved corrugated steel pipe (ACPCSP), plain aluminum (AL), and polyethylene (PE) pipe. All of these alternatives are structurally adequate for the design load. A 24-in. diam smooth wall pipe will carry the design flow at the design slope of 1 percent. A 27-in. diam pipe will be required for the corrugated alternatives because of their higher n value. The differential escalation rate is projected to be zero for installation costs and for the concrete, aluminum, and plain galvanized materials. A rate of 3 percent will be assumed for the total cost the asphalt coated and paved corrugated steel and polyethylene pipes to account for expected increases in the cost of petroleum and natural gas, respectively. Assume that an exception will be granted to allow a 50-year analysis period, that maintenance costs over the analysis period are equal for all alternatives, that the facility is to be abandoned at the end of the analysis period, and that pipe still serviceable at the end of the analysis period will not be

recovered for reuse or resale (no salvage value). Uncertainty analysis will be omitted for simplicity. The costs stated herein are hypothetical costs. They do not apply to any particular project, do not reflect current market prices, and are not to be used for an actual LCCA.

74. From Equation 9, the expected service life of reinforced concrete pipe is about 80 years. It should therefore last through the entire analysis period. The current cost is \$12.50/ft, delivered, plus \$10.00/ft for installation. Since e=0 for both materials and installation, the one-time cost to be incurred in 2 years is simply 12.50 + 10.00 = \$22.50/ft, in terms of today's dollars. The PW is \$18.59/ft.

75. Since the pH is near the environmental limits specified in paragraph 7 for plain galvanized pipe, Equation 2 should be used to estimate the service life of that alternative. For a pH of 6.0 and a minimum resistivity of 6,000 ohm-cm, a 16 gage, plain galvanized CSP has an expected life of about 25 years. This alternative will require a replacement at the midpoint of the analysis period. The current cost of 27 in. plain galvanized pipe is \$10.65/ft, delivered, including bands, plus \$8.50/ft for installation, for both initial construction and replacement. Since e=0 for both materials and installation, the cost to be incurred in 2 years and again in 27 years is 10.65 + 8.50 = \$19.15/ft. The PW of the initial installation is 19.15 $(1/1.1)^2$ = \$15.82/ft. The PW of the replacement is \$19.15 $(1/1.1)^{27}$ = \$1.46/ft. The total PW for this alternative is thus 15.82 + 1.46 = \$17.28/ft. All these are expressed in terms of today's dollars.

76. Asphalt coating and paving can be used to extend the life of plain galvanized pipe. Assume that this coating will add 25 years to the life of the pipe. The service life of an ACPCSP at this site will be 25 + 25 = 50 years, and no replacement is anticipated during the analysis period. The current cost for ACPCSP is \$13.90/ft, including bands. Assuming a 3 percent annual differential escalation rate due to the cost of the asphalt, the pipe will cost 13.90 × $(1.03)^2$ = \$14.75/ft at the time of installation. Installation is currently \$9.50/ft. Assuming e=0 for installation, this cost will remain at \$9.50/ft. The total cost of this alternative will thus be 14.75 + 9.50 = \$24.25/ft. The PW is 24.25 $(1/1.1)^2$ = \$20.04/ft.

77. The proposed AASHTO design procedure (Appendix A) is structured to provide a 50-year service life. One 24-in. smooth-flow PE pipe meeting the requirements of this procedure costs \$16.50/ft. An escalation of 3 percent

for 2 years yields a cost at time of installation of $16.50 \times (1.03)^2$ = \$17.50/ft. Installation is and will be (e=0) \$8.00/ft. At the time of installation, the total cost will be 17.50 + 8.00 = \$25.50/ft. The PW is $25.50 \times (1/1.1)^2$ = \$21.08/ft.

78. This site is within the environmental limits for aluminum pipe; therefore, a life in excess of the required 50 years can be expected. The current cost for aluminum pipe is \$11.90/ft, including bands. Installation is \$8.00/ft. Since the differential escalation is zero for both material and installation, the future cost will be 11.90 + 8.00 = \$19.90. The PW is $19.90 \times (1/1.1)^2$ = 16.44/ft.

79. The life cycle cost of these alternatives is summarized below:

Cost	24 in. RCP	27 in. CSP 1st	27 in. CSP 2nd	27 in. CSP Total	24 in. ACPCSP	24 in. PE	27 in. AL
Current Material	12.50	10.65	10.65		13.90	16.50	11.90
Installation (e=0)	10.00	8.50	8.50		9.50	8.00	8.00
Escalated Material	12.50	10.65	10.65		14.75	17.50	11.90
PW Material	10.33	8.80	0.81	9.61	12.19	14.47	9.83
PW Installation	8.26	7.02	0.65	7.67	7.85	6.61	6.61
Total PW	18.59			17.28	20.04	21.08	16.44
Choice	3			2	4	5	1

In this example, plain corrugated aluminum pipe (AL) would be chosen for its lowest LCC. If two or three alternatives are to be selected as bid options, then AL and CSP or AL, CSP, and RCP would be considered.

Summary

80. The LCC of drainage structures is determined according to the criteria in TM 5-802-1. Because of the nature of drainage structures, an analysis period greater than 25 years may be justified. The alternatives are order ranked by LCC, and the alternative with the lowest LCC is selected. Uncertainty in the true costs and tie-breaking criteria are addressed in TM 5-802-1.

PART IV: CONCLUSIONS

81. The LCC of a drainage structure design alternative is the estimated total cost of that design. Except for determining a service life for the various types (materials) of drainage structures, the procedures for LCCA are well established. The guidelines presented in Part II of this report can be used to estimate the service life of a particular design or to ensure a 50-year service life. Thus, the procedures for economic analysis described in TM 5-802-1 can be used to determine LCC. While the LCC is only one of the decision factors used to select the preferred design alternative from among the feasible alternatives, it is generally the most important. The importance of the other decision factors are established by the minimum functional requirements of the project. The alternatives can then be order ranked by LCC, and the best design can be rationally and confidently selected.

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APPENDIX A

PROPOSED AASHTO DESIGN PROCEDURE

SECTION 18

SOIL - THERMOPLASTIC PIPE INTERACTION SYSTEMS

18.1 GENERAL

18.1.1 Scope

The specifications of this section are intended for the structural design of plastic pipes. It must be recognized that a buried plastic pipe is a composite structure made up of the plastic ring and the soil envelope, and that both materials play a vital part in the structural design of plastic pipe.

18.1.2 Notations

A = required wall area (Article 18.2.1)

A = area of pipe wall (Article 18.3.1)

B = water bouyancy factor (Articles 18.2.2 and 18.3.2)

c = distance from inside surface to neutral axis (Articles 18.2.2, 18.3.2 and 18.4.2)

D_e = effective diameter = ID + 2c

E = modulus of elasticity of pipe material (Articles 18.2.2 and 18.3.2)

FF = flexibility factor (Articles 18.2.3 and 18.3.3)

f_a = allowable stress-specified minimum tensile strength divided by safety factor (Article 18.2.1)

f_{cr} = critical buckling stress (Articles 18.2.2 and 18.3.2)

f_u = specified minimum tensile strength (Articles 18.2.1, 18.3.1 and 18.3.2)

I = moment of inertia, per unit length, of cross section of the pipe wall (Articles 18.2.2 and 18.3.2)

ID = inside diameter (Articles 18.2.2, 18.3.2 and 18.4.2)

M_s = soil modulus (Articles 18.2.2, 18.3.2 and 18.4.2)

OD = outside diameter (Article 18.4.2)

P = design load (Article 18.1.4)

SF = safety factor (Article 18.2.1)

T = thrust (Article 18.1.4)

T_L = thrust, load factor (Article 18.3.1)

T_s = thrust, service load (Article 18.2.1)

o = capacity modification factor (Article 18.3.1)

18.1.3 Loads

Design load, P, shall be the pressure acting on the structure. For earth pressures see Article 3.20. For live load see Articles 3.4 to 3.7, 3.11, 3.12 and 6.4, except that the words "When the depth of fill is 2 feet or more" in Article 6.4.1 need not be considered. For loading combinations see Article 3.22.

18.1.4 Design

18.1.4.1 The thrust in the wall shall be checked by two criteria. Each considers the mutual function of the plastic wall and the soil envelope surrounding it. The criteria are:

(a) Wall area

(b) Buckling stress

18.1.4.2 The thrust in the wall is:

$$T = P \times \frac{D}{2} \qquad (12\text{-}1)$$

where

P = design load, in pounds per square foot;

D = diameter in feet;

T = thrust, in pounds per foot.

18.1.4.3 Handling and installation strength shall be sufficient to withstand impact forces when shipping and placing the pipe.

18.1.5 Materials

The materials shall conform to the AASHTO and ASTM specifications referenced herein.

18.1.6 Soil Design

18.1.6.1 Soil Parameters

The performance of a flexible culvert is dependent on soil structure interaction and soil stiffness.

The following must be considered:

(a) Soils

(1) The type and anticipated behavior of the foundation soil must be considered; i.e., stability for bedding and settlement under load.

(2) The type, compacted density, and strength properties of the soil envelope immediately adjacent to the pipe must be established.

Good side fill is obtained from a granular material with little or no plasticity and free of organic material, i.e., AASHTO classification groups A-1, A-2, and A-3, compacted to a minimum 90 percent of standard density based on AASHTO Specifications T99 (ASTM D698).

(3) The density of the embankment material above the pipe must be determined. See Article 6.2.

(b) Dimensions of soil envelope

The general recommended criteria for lateral limits of the culvert soil envelope are as follows:

(1) Trench installations - 2 feet minimum each side of culvert. This recommended limit should be modified as necessary to account for variables such as poor in-situ soils.

(2) Embankment installations - one diameter each side of culvert.

(3) The minimum upper limit of the soil envelope is one foot above the culvert.

18.1.7 Abrasive or Corrosive Conditions

Extra thickness may be required for resistance to abrasion. For highly abrasive conditions, a special design may be required.

18.1.8 Minimum Spacing

When multiple lines of pipes greater than 48 inches in diameter are used, they shall be spaced so that the sides of the pipe shall be no closer than one-half diameter or 3 feet, whichever is less, to permit adequate compaction of backfill material. For diameters up to and including 48 inches, the minimum clear spacing shall not be less than 2 feet.

18.1.9 End Treatment

Protection of end slopes may require special consideration where backwater conditions may occur, or where erosion and uplift could be a problem. Culvert ends constitute a major run-off-the-road hazard if not properly designed. Safety treatment, such as structurally adequate grating that conforms to the embankment slope, extension of culvert length beyond the point of hazard, or provision of guardrails, are among the alternatives to be considered. End walls on skewed alignment require a special design.

18.1.10 Construction and Installation

The construction and installation shall conform to Section 23 - Division II.

18.2 SERVICE LOAD DESIGN

Service Load Design is a working stress method, as traditionally used for culvert design.

18.2.1 Wall Area

$$A = T_s/f_a$$

where

- A = required wall area in square inches per foot
- T_s = thrust, service load in pounds per foot
- f_a = allowable stress-specified minimum tensile strength, pounds per square inch, divided by safety factor, f_u/SF.

18.2.2 Buckling

Walls with the required wall area, A, shall be checked for possible buckling. If the allowable buckling stress, f_{cr}/SF, is less than f_a, the required area must be recalculated using f_{cr}/SF in lieu of f_a. The formula for buckling is:

$$f_{cr} = 0.77\ (12R/A)\ \sqrt{BM_s\ EI/0.149^3}$$

where

- B = water buoyancy factor
  = $1-0.33h_w/h$
- h_w = height of water surface above top of pipe
- h = height of ground surface above top of pipe
- E = Long term (50-year) modulus of elasticity of the plastic in pounds per square inch;
- M_s = soil modulus in pounds per square inch
  = 1700 for side fills meeting Article 18.1.6
- f_{cr} = critical buckling stress in pounds per square inch
- R = effective radius
  = $c + ID/2$
- I = moment of inertia of the pipe wall per unit length of cross section, in^4/in.

18.2.3 Handling and Installation Strength

Handling and installation rigidity is measured by a flexibility factor. FF, determined by the formula

$$FF = D_e^{2}/EI$$

where

FF = flexibility factor in inches per pound;

D_e = effective diameter in inches;

E = modulus of elasticity of the pipe material in pounds per square inch;

I = moment of inertia per unit length of cross section of the pipe wall in inches to the 4th power per inch.

18.3 LOAD FACTOR DESIGN

Load Factor Design is an alternative method of design based on ultimate strength principles.

18.3.1 Wall Area

$$A = T_L/of_u$$

where

A = area of pipe wall in square inches per foot;

T_L = thrust, load factor in pounds per foot;

f_u = specified minimum tensile strength in pounds per square inch;

o = capacity modification factor.

18.3.2 Buckling

If f_{cr} is less than f_u, A must be recalculated using f_{cr} in lieu of f_u. The formula for buckling is:

$$f_{cr} = 0.77\ (12R/A)\sqrt{BM_s\ EI/0.149^3}$$

where

B = water buoyancy factor

= $1-0.33h_w/h$

h_w = height of water surface above top of pipe

h = height of ground surface above top of pipe

E = Long term (50-year) modulus of elasticity of the plastic in pounds per square inch;

M_s = soil modulus in pounds per square inch

= 1700 for side fills meeting Article 18.1.6

f_{cr} = critical buckling stress in pounds per square inch;

R = effective radius

= $c + ID/2$

I = moment of inertia of the pipe wall per unit length of cross section, in^4/in.

18.3.3 Handling and Installation Strength

Handling rigidity is measured by a flexibility factor, FF, determined by the formula

$$FF = D_e^{\,2}/EI$$

where

FF = flexibility factor in inches per pound;

D_e = effective diameter or maximum span in inches;

E = modulus of elasticity of the pipe material in pounds per square inch;

I = moment of inertia per unit length of cross section of the pipe wall in inches to the 4th power per inch.

18.4 PLASTIC PIPE

18.4.1 General

18.4.1.1 Plastic pipe may be smooth wall, corrugated or externally ribbed and may be manufactured of polyethylene (PE) or poly (vinyl chloride) (PVC). The material specifications are: