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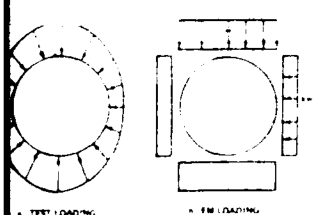

TECHNICAL REPORT SL-80-4

STRENGTH DESIGN OF REINFORCED CONCRETE HYDRAULIC STRUCTURES

Report 9

ANALYSIS AND DESIGN OF REINFORCED CONCRETE CONDUITS

by

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The purpose of this report is to indicate ways of updating current Corps of Engineers specifications for the analysis and design of reinforced concrete culverts.

To this end, after reviewing the current state of the art in conduit design, attention is directed to the possibility of including soil-structure interaction in conduit analysis and design. By analysis and example, the beneficial effects of soil-structure interaction are demonstrated.

A draft for an updated set of specifications for design of concrete condutis, based on the 1988 AASHTO Specifications, is presented. These strength-design specifications are then used in conjunction with soil-structure interaction analysis to design a number of conduits of various shapes. The resulting designs indicate that considerable savings may be possible by reliance on thinner-walled sections.

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19. ABSTRACT (Continued).

Lastly, questions that need additional attention before definite answers can be supplied are outlined and discussed.

PREFACE

This study was conducted during the period March through August 1988 by the Department of Civil, Environmental, and Architectural Engineering (CEAE), University of Colorado, under the sponsorship of Headquarters, US Army Corps of Engineers (HQUSACE). The Technical Monitor was Dr. Tony Liu.

This work was monitored by the US Army Engineer Waterways Experiment Station (WES), Structures Laboratory (SL), under the supervision of Messrs. Bryant Mather, Chief, James T. Ballard, Assistant Chief, and Dr. Jimmy P. Balsara, Chief, Structural Mechanics Division (SMD). Mr. Stanley C. Woodson, MSD, and Dr. Robert L. Hall, SMD, monitored the study, with Mr. Woodson coordinating the publication of the work with the Information Technology Laboratory, WES.

Dr. Kurt H. Gerstle, CEAE, was the Principal Investigator. He was assisted by Messrs. Piotr Noakowski and Thomas Rauscher, CEAE.

Acting Commander and Director of WES during preparation of this report was LTC Jack R. Stephens, EN. Dr. Robert W. Whalin was Technical Director.

CONTENTS

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

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

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	25.4	millimetres
kips (force)	4.448222	kilonewtons
kip-inches	112.9848	newton-metres
kips (force) per inch	0.1751269	kilonewtons per millimetre
kips (force) per square inch	6.894757	kilopascals
kips (force) per square foot	47.88026	megapascals
pounds per square inch	0.006894757	megapascals
square inches	6.4516	square centimetres

STRENGTH OF REINFORCED CONCRETE HYDRAULIC STRUCTURES

ANALYSIS AND DESIGN OF REINFORCED CONCRETE CONDUITS

PART I: INTRODUCTION

Purpose and Goal

1. Corps of Engineers guidelines for the design of reinforced concrete conduits are currently contained in Engineer Manual EM 1110-2-2902, dated 1969. This manual represents the state of the art of the 1960's; in the intervening twenty years, much progress has been made, both in understanding the physical phenomena and in their application to structure design using electronic computers.

2. In recognition of these developments, the Corps of Engineers has begun to update its design methods for hydraulic structures: two reports (1-1, 1-2) introduce the strength method of reinforced concrete design which has generally supplanted working stress methods in structural concrete practice, and several studies (1-3, 1-4, 1-5) have laid the groundwork for a similar move for circular reinforced concrete pipe.

3. This report is intended to continue this work with suggestions for the design of reinforced concrete culverts of general shape. In order to do justice to the actual behavior of the structure in the ground, soil-structure interaction will be considered. Rather than considering the soil only as the load, that is, the problem, it should also be considered as support, that is, part of the solution. As the pipe deforms, it will be constrained by the surrounding soil, and the more it deforms, the greater the support. The soil should therefore be considered part of the structure, and analyzed accordingly.

The resulting thinner, more flexible structures should lead to savings compared to current rigid designs which neglect the supporting role played by the surrounding soil medium.

4. These approaches will be stressed in this report, because they promise the greatest improvement in design methods. Another suggested advance will be in the proposed use of the strength method of design to replace the older working stress method.

5. It should be noted that a good deal of this work has already been done by the Corps of Engineers, and that these contributions should not be overlooked; for instance, the Harter and Bircher report of 1980 (1-6) provides a beautiful analysis-design tool of great usefulness when fully implemented and used.

6. The purpose of this study then is to outline some of the mentioned approaches, to propose some procedures for their implementation using modern computer methods, and to attempt their organization in a design manual for design office practice. The scope of this project will not permit completion of any of this work. The aim, rather, is to provide a skeleton which will be fleshed out by further work directed at specific problems which are only touched within this study.

Contents of this Report

7. With the above goals in mind, the contents of this report are organized in the following sequence: In Part II, some of the currently used analysis and design methods are reviewed and critically appraised, in order to identify problem areas and avenues for improvement.

8. Part III considers the problem of soil-structure interaction analysis; a relatively simple, computer-based approach is presented. implemented, and used in several examples to illustrate typical results and general trends.

9. Part IV addresses the structural design side of conduit engineering. It consists of two parts. First, a draft set of specifications is presented, complete with commentary and discussion of areas where further research is needed before these specifications can be finalized. The second part consists of a computer program which implements these draft specifications, and an example illustrating its use.

10. Part V applies both the soil-structure analysis of Part III and the design specifications of Part IV in order to carry out design of some sample conduit sections of several non-circular shapes. These designs are then used to highlight the effects of soil-structure interaction, and to arrive at recommendations for optimal design of conduits under ground. Part VI, finally, summarizes our approach, findings, and conclusions.

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PART II: THE STATE OF THE ART

Introduction

11. This short review of reinforced concrete pipe design practice highlights the impending transition from the traditional thick-walled conduit design to thinner sections which tend to rely upon support from the surrounding soil for their stability. To this end, we have subdivided our state-of-the-art report into three parts:

1. The traditional methods which result in thick-walled conduits which do not rely on soil-structure interaction for strength and serviceability.
2. The more modern approaches which are derived from field measurements to assess the effects of soil-structure interaction.
3. The modern analytical methods which use the finite-element method, or other discrete models, to determine the effects of soil-structure interaction in a rational manner.

Lastly, some critical comments are added, with suggestions for possible directions for further work.

12. We consider that the choice of analysis assumptions according to this outline is decisive for the basic issue of "thick versus thin" conduit design. The choice of design procedure - whether working stress or strength method - will be of lesser importance in its effect on the designed structure.

Traditional Methods

The Three-Edge Bearing Test

13. Traditionally, culvert pipe has been designed according to empirical methods based on the three-edge bearing test as specified in Ref. 2-1. The result of such tests, the so-called D-load, defined by the cracking, or

ultimate test load per unit length of pipe, divided by the pipe diameter, presumably will predict the vertical soil pressure capacity of the pipe. Ref. 2-2 specifies required D-loads for different strength classes and provides design tables based on these requirements. Three different thickness classes, A, B, and C, of dimension ratio diameter/wall thickness about 12, 10, and 8 are listed. As will be seen later, all of these sections can be considered "thick-walled" or "rigid".

14. Similar provisions in ASTM (2-3, 2-4) for conduits of horseshoe and elliptical section base the strength determination on results of three-edge bearing tests. All of these designs will lead to thick-walled or rigid pipes which will ignore possible support from lateral soil pressures.

American Concrete Pipe Association (ACPA) Approach

15. The "indirect method" of the ACPA (2-5) follows the D-load method of ASTM, but accounts for the lateral support provided by the backfill by the "bedding factor" which serves to reduce the effective vertical load and therefore the required D-load on the pipe.

16. Because neither pipe wall thickness nor soil stiffness are explicitly involved in the determination of the bedding factor, this cannot be considered a rational way of accounting for soil-structure interaction; we therefore list it among the methods leading to thick-walled sections.

The Bureau of Reclamation Approach

17. The Bureau of Reclamation uses the Olander formulation (2-6) for pressure distribution for the analysis of circular pipe sections, consisting of assumed cosinusoidal radial pressure varying from a maximum at the crown to zero at the edge of the bedding. This load is equilibrated by a similar

cosinusoidally varying reaction pressure extending over the bedding angle. This pressure distribution is based on the classical measurements of Marston and Spangler (2-7) which were obtained on thick pipes. Since soil and structure stiffnesses do not enter this load specification, it cannot take soil-structure interaction into account.

18. Working-stress or strength design has been used by the Bureau of Reclamation for design. Tables for circular pipe (2-8) show diameter-wall thickness ratios no greater than 12, thus classifying these conduits as "rigid" or "thick-walled".

The Corps of Engineers Approach

19. In Engineer Manual EM 1110-2-2902 (2-9) the load determination is specified explicitly for rigid culvert sections, that is, soil-structure interaction is excluded. Soil pressures are a uniform vertical pressure, following the approach of Paris (2-10), and a linearly varying lateral pressure related to the vertical load by a specified lateral load coefficient which depends on the backfill conditions.

20. These simple loadings do not consider the flexibility of the conduit structure, nor specific bedding conditions. Linearly-elastic analysis is performed to determine internal forces, and section design is carried out using working stress theory of reinforced concrete. The very low allowable stresses and conservative design procedure insure that cracking does not become a problem in these structures.

21. To implement these methods in the design office, Harter, Bircher, and Wilson (2-11) have written a computer program capable of analyzing and designing the reinforcement for conduit sections such as those shown in Fig. 2-1.

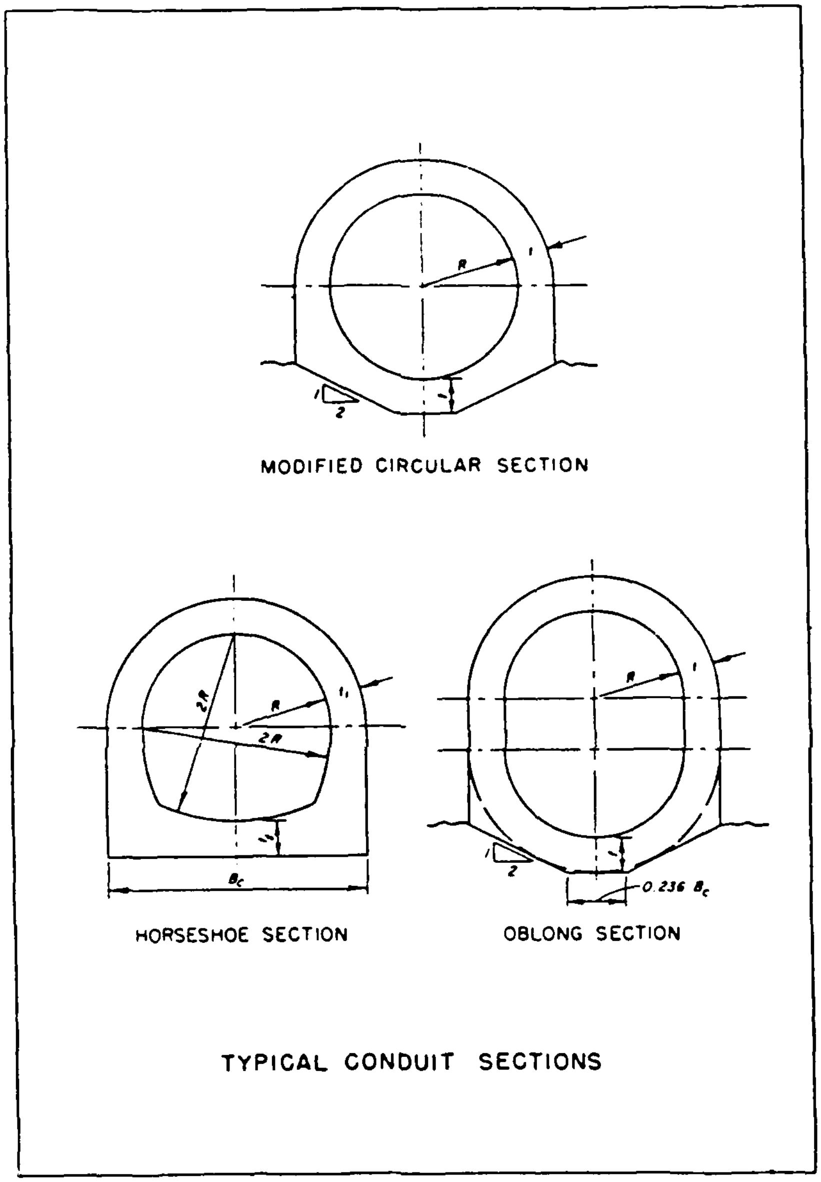

Fig. 2-1. Thick-walled conduit sections

Analysis of the two-times indeterminate structure is by the force method, and design is according to the working stress provisions of ACI 318-63 (2-12).

22. This appears to be an excellent program for design of rigid conduits, but it cannot handle soil-structure interaction. If this effect is to be considered, the vastly increased degree of indeterminacy suggests use of the displacement, rather than the force method, for analysis.

23. More recently, studies have been initiated by the Corps of Engineers toward adoption of the strength design method for hydraulic concrete structures (2-13).

Empirical Approach to Soil-Structure Interaction

24. Australian experience with thin-walled concrete pipe, and extensive field testing of experimental installations of thin-walled pipe by the California Department of Transportation (2-14) and Hydro-Conduit Corporation (2-15) led to the concept of developing flexible conduit sections capable of mobilizing passive soil pressures to help support the pipe.

25. While the interpretation of the CALTRANS field tests led to differing conclusions (2-16), nevertheless these data resulted in some very clear-cut design recommendations involving the "dimension ratio" (DR) diameter/wall thickness as a primary variable. Circular pipes are divided into three classes, as shown in Fig. 2-2 (2-14): "rigid" of DR less than 12, "semi-rigid", of DR between 12 and 20, and "flexible", of DR larger than 20. For each class, the soil pressure distribution is specified as in Fig. 2-2, ranging from a lateral/vertical pressure ratio of .3 for rigid to 1.0, or hydrostatic, for flexible pipe. According to this scheme, all conduits

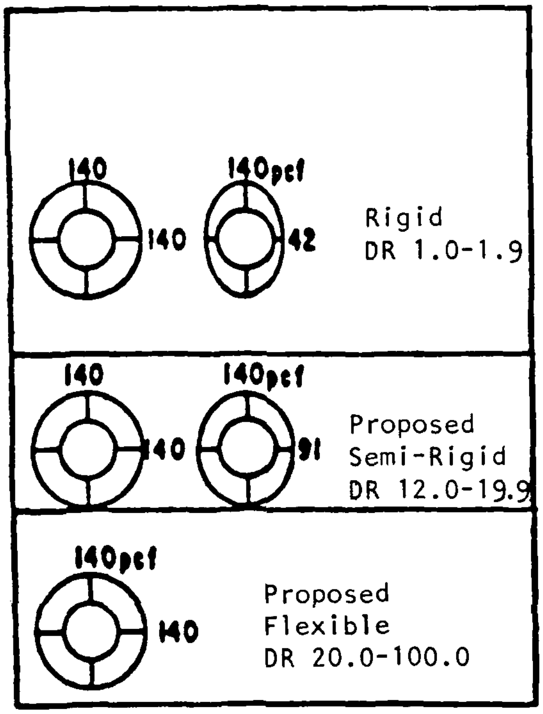

Fig. 2-2. Pipe classes by dimension ratio and recommended soil pressures

designed according to the traditional methods are rigid pipe sections. In contrast, one recommendation of Ref. 2-15 calls for a minimum DR of 16 for all circular pipe.

26. Obviously, the redistribution of soil pressures in the semi-rigid and flexible pipes depends on longitudinal cracking of the concrete. The effect of these cracks deserves close attention.

27. These concepts lead to very simple analysis procedures, they are, however, restricted by their empirical basis. More general, and generally accepted, data and verification are needed before they can be accepted for routine design.

Analytical Methods for Soil-Structure Interaction

28. Analytical prediction of soil-structure interaction, supported by reliable testing and field measurements, appears the most appropriate basis for rational conduit design, since it permits unlimited variation of parameters and conditions. Once verified, its results could be used to formulate design aids for routine office practice.

29. An early finite-element program for conduit-soil analysis is CANDE (Culvert ANalysis and DEsign) (2-17). This program can handle elastic or inelastic structure and soil and an unlimited variety of soil-structure combinations, and some interesting results are presented in Ref. 2-17. This program did not find widespread use in practice. Whether this is due to lack of distribution or support. user-unfriendliness, unreliability, or other factors is hard to determine.

30. A more recent effort along similar lines is SPIDA (Soil-Pipe Interaction Design and Analysis) (2-18). Extensive verification is cited (2-19),

documenting the validity of its predictions during the developmental stages of the program. Its capabilities include non-linear soil and structure characterization, arbitrary geometries, and analysis, as well as flexural and shear reinforcement design. This program has been used on many occasions and is currently being converted from main frame to PC use. ACPA intends to distribute this program to designers, so there is hope that this program will avoid the premature obscurity of its predecessors.

Comments

31. The traditional methods, having served nobly in times of lesser analytical capabilities, appear obsolete now. Our understanding of soil behavior, structure behavior, and their interaction has sufficiently advanced so that less reliance needs to be be placed on purely experimental approaches such as the D-load test. To quote an early paper on the subject (2-20), "In any soil-structure system, it is the combination of the soil and the structure which provides the supporting capability, and it is inappropriate to devise load tests for the structural element (only)". Indeed, any design not based on the ratio of soil-to-structure stiffness should be suspect as leading to unduly rigid and uneconomical designs.

32. It is interesting to observe that the empirical and analytical approaches seem to have developed almost independently of each other. A much closer tie between these is desirable, with analysis results and field measurements complementing each other like hand-in-glove. The design approach based on lateral-to-vertical pressure ratios as function of the dimension ratio appears simple and possibly effective. It should be verified analytically for a range of conditions.

33. Program SPIDA might serve as an important tool in WES conduit research for such tasks, as well as for verification of the results presented in the current report using simpler tools.

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PART III: SOIL-STRUCTURE INTERACTION IN CONDUIT DESIGN

Introduction

34. American concrete conduit design practice has in the past neglected the beneficial effects of the surrounding soil. Soil pressures have been considered only as the problem, not as part of the solution. This approach leads to thick-walled conduits, as shown by the typical sections of Fig. 3-1 (3-1).

35. In contrast, metal culvert design practice considers the soil surrounding the pipe as an integral part of the structure. Only through this soil support can these culverts be built as thin as the steel sections shown in Fig.3-2 (3-2).

36 The contrast between the wall thicknesses of the sections of Figs. 3-1 and 3-2 reflects the fact that the structure tries to accommodate the design assumptions: the rigid sections of Fig. 3-1 will actually ignore the soft soil, whereas the flexible sections of Fig.3-2 will deform sufficiently to activate restraining soil pressures. The more flexible the structure as compared to the soil, the greater the role played by the soil in supporting the loads.

37. In recent years, designers of concrete conduits have also begun to consider soil-structure interaction, in the hope of achieving savings through use of thinner-walled pipe. Two different approaches have been taken toward this goal:

1. Empirical Approach (3-3, 3-4): From field measurements of pipe deformations and strains under ground, effective soil pressures are computed and reduced to lateral earth pressure coefficients for different pipe

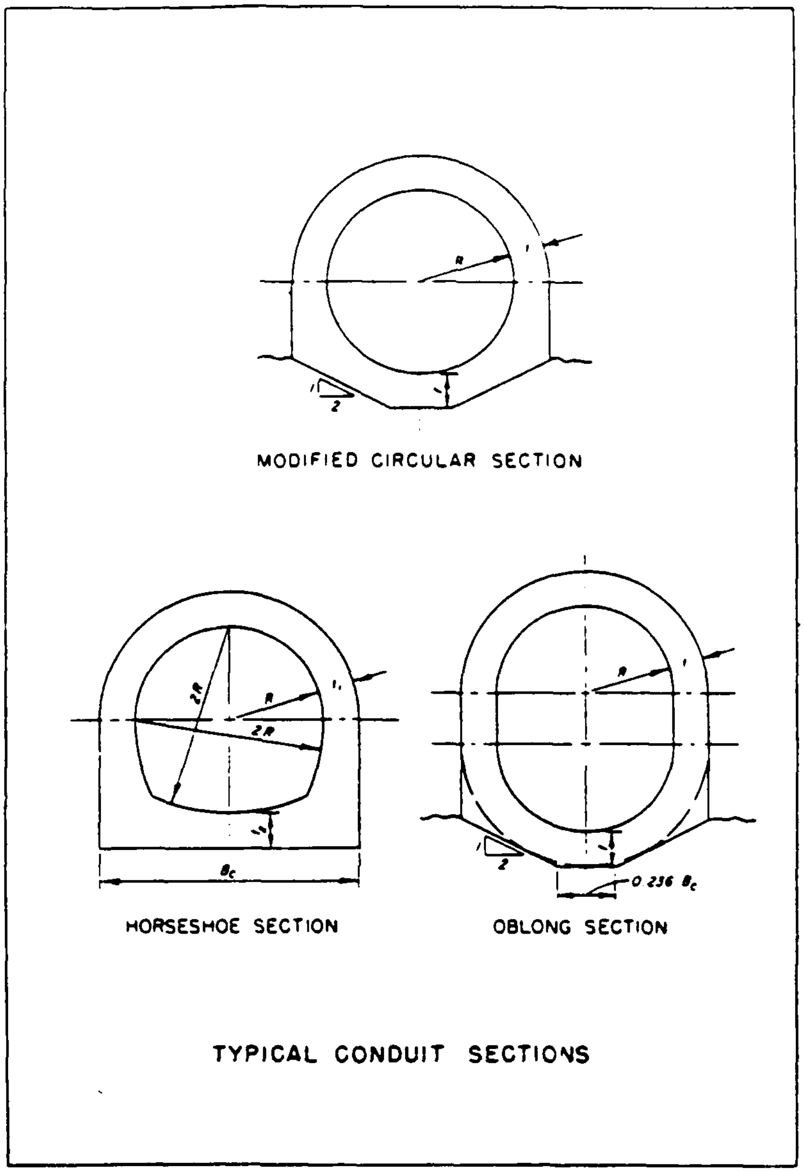


Fig. 3-1. Thick-walled culvert sections

Fig. 3-2. Thin-walled culvert sections

dimensions and bedding conditions. The predominant design parameter in this approach is the "dimension ratio" diameter/wall thickness.

2. Analytical Approach (3-5, 3-6, 3-7): This approach is carried out by rational analysis such as the finite-element method, which considers realistic material properties in a structure consisting of the concrete culvert and a sufficiently extensive portion of the surrounding soil. Analyses for dead and superimposed loads lead directly to internal forces which can be used in design.

38. The empirical method is simple to use for design but is limited by the scant, and sometimes questionable, field data which form its basis. The analytical approach can handle a variety of conditions in a rational manner, but leads to difficult non-linear analyses requiring a great deal of input information, and great care in the interpretation of both input and output data.

39. In the following, we will explore the issue of "thick versus thin" in the design of buried reinforced concrete conduits by means of a feasibility study to determine whether concrete sections of adequate strength and serviceability can be designed sufficiently thin so as to activate soil support. To this end, a simple model for soil-structure interaction is presented in the next section, and idealized stiffness properties of soil and structure are discussed in the following. Next, linear and non-linear analysis methods are shortly outlined, and applied to the determination of internal forces under various assumptions and conditions. Results are presented in a form which sheds light on the beneficial influence of soil support and permits conclusions for design.

40. Because this is a feasibility study showing general trends, the models are kept as simple as possible. The various assumptions made here are not necessarily recommended for final design.

Soil and Structure Properties

Soil-Structure Interaction

41. Consider a culvert section embedded in soil, as shown in Fig. 3-3. If the pipe deforms under applied load, as shown dashed, the bulging portions will bear against the surrounding soil. Passive soil pressure generated will depend on the deflection, which is a function of the structure stiffness, and on the soil stiffness. The greater the ratio of soil to structure stiffness, the greater the soil reactions. A clear picture of the stiffnesses is therefore essential.

Soil Stiffness

42. The following assumptions on soil behavior are made in this study:

1. Linearly-elastic soil behavior of uniaxial stiffness, or subsoil modulus, k, ranging from zero to 0.3 kips/in^3 (82.5 MN/m^3).
2. Only uniaxial soil restraint normal to the conduit surface is considered. Friction between soil and concrete is neglected. No tension can occur between soil and concrete.

This idealization corresponds to that of Winkler (3-9) and can be represented analytically by a bed of either continuous or discrete elastic springs, as shown in Fig. 3-3 by the soil, and in Fig. 3-4 by radial elastic bars.

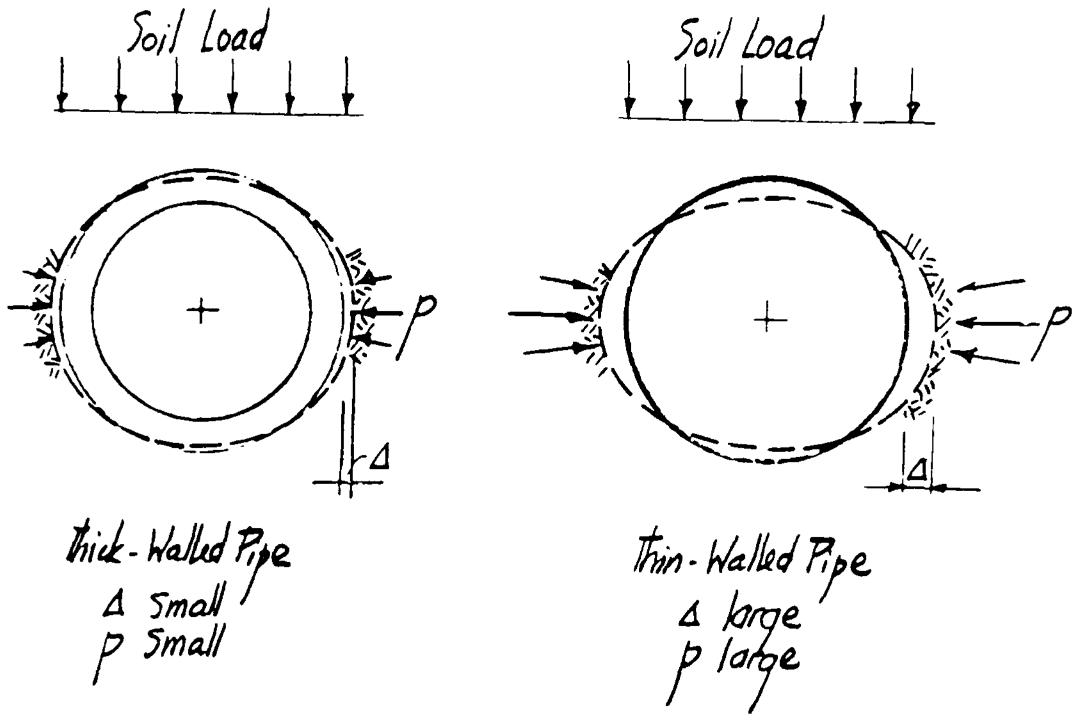


Fig. 3-3. Soil structure interaction

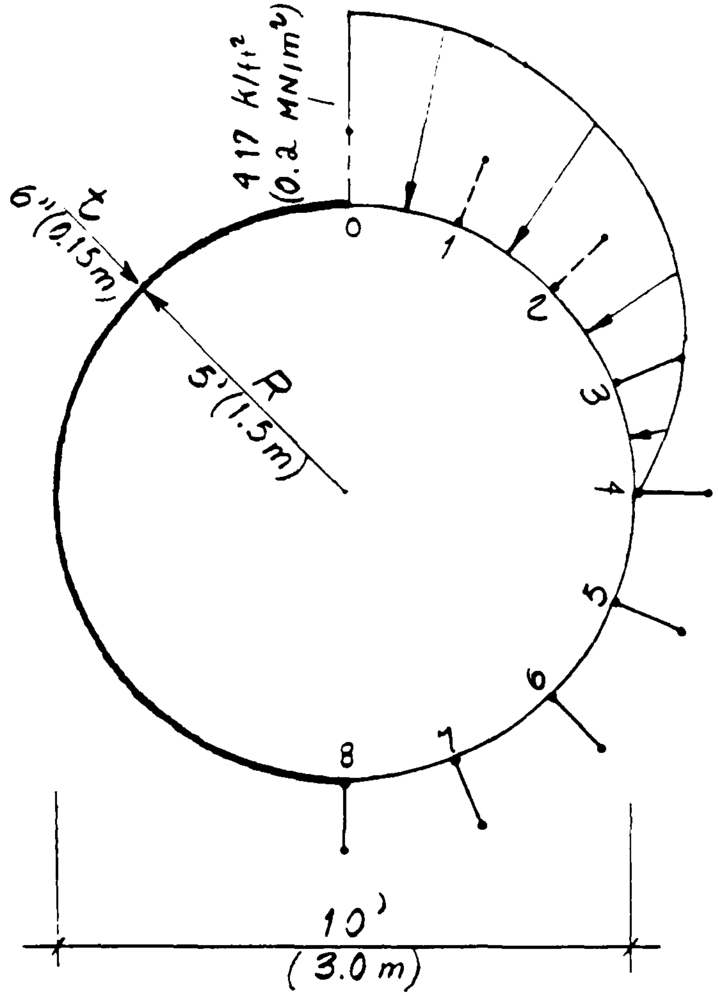


Fig. 3-4. Framed-structure idealization of soil-structure interaction, circular pipe

Structure Stiffness

43. The stiffness of the ring section depends on the cross-sectional bending stiffness EI, in which E is the elastic modulus of the concrete, and I is the moment of inertia of the transformed uncracked section for moments below the cracking moment M_c (which depends also on the axial compression force).

44. The cracking moment M_c under axial force N is computed by elastic theory. For moments larger than the cracking moment, I is the moment of inertia of the cracked section, which can be calculated according to Sec.9 of ACI 318-83 (3-10), for pure bending, or according to Ref. 3-11, which considers the sudden increase of curvature due to crack opening, the effects of tension-stiffening due to steel-concrete bond between cracks, and the presence of axial compression.

45. The formulation of Ref. 3-11 leads to the relation between proportionally increasing moment and axial force $M/N = e =$ constant and the resulting average curvature shown in Fig. 3-5. According to these curves, the uncracked stiffness EI_g is independent of the axial force. The cracking moment M_c depends on the excentricity e, as does the sudden curvature increase at M_c. The stiffness after cracking depends on the moment-force ratio e, ranging between the limiting values of EI_g and EI_{cr}.

Analysis

Analytical Approach

46. The case of a circular elastic ring of cross-sectional stiffness EI and radius R, embedded in an elastic Winkler medium of modulus k capable of taking both tension and compression, has been discussed by Hetenyi (3-12). The

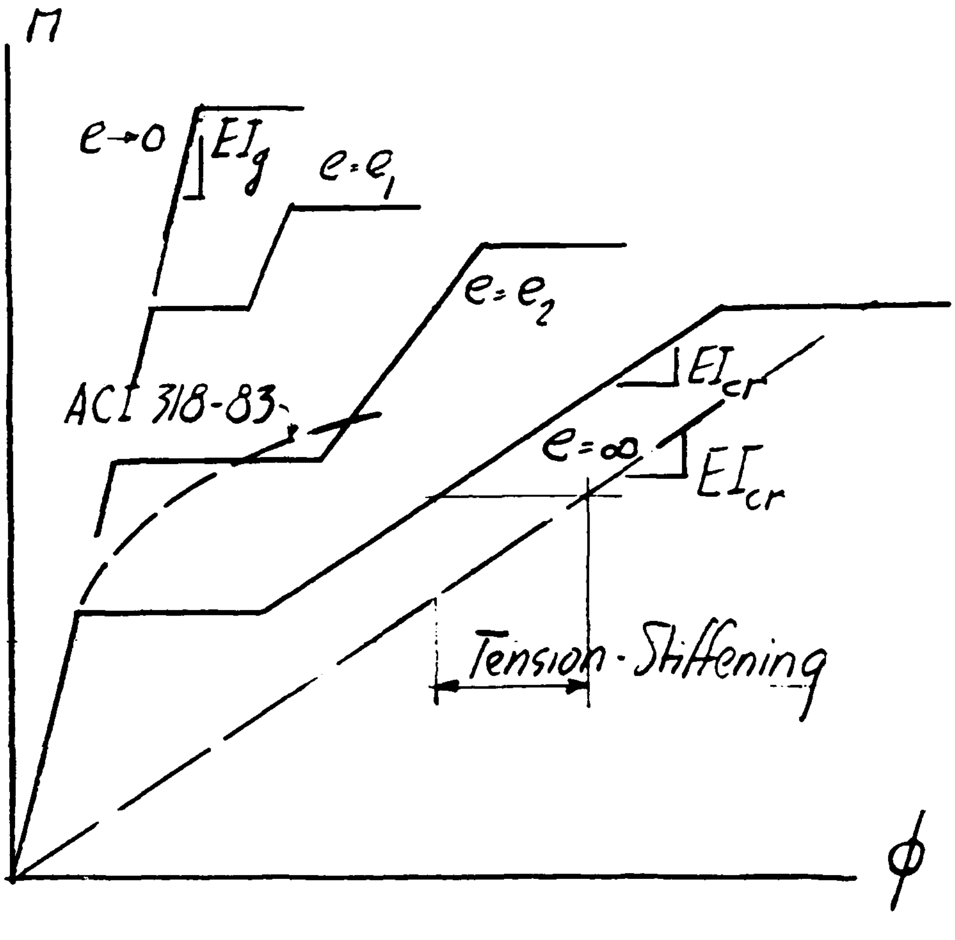


Fig. 3-5. Moment-thrust curvature relations

differential equation for the radial displacement y as function of the angle is given there as

$$\frac{d^5y}{d\theta^5}+2\frac{d^3y}{d\theta^3}+\left(\frac{1+kR^4}{EI}\right)\frac{dy}{d\theta}=0 \tag{1}$$

47. The only significant structure property is the non-dimensional stiffness ratio

$$\frac{kR^4}{EI}=\gamma \tag{2}$$

We will use this parameter in what follows as an indicator of soil-structure interaction. In particular, for the case of a rectangular uncracked gross section of unit length, Eq.2 becomes

$$\gamma=\frac{12k}{E}\cdot R\left(\frac{R}{t}\right)^3 \tag{3}$$

This shows that the response is strongly dependent on the "dimension ratio" of pipe diameter to wall thickness.

48. The solution of Eq.1 is subject to limitations of geometry and linearity which make it unsuitable for practical analysis of conduits of general shape. We will therefore resort to a numerical approach.

Numerical Approach

49. In our numerical approach, the culvert section is discretized into a number of straight segments of appropriate stiffness, separated by nodal points, as shown in Fig. 3-4. The soil constraint is represented by an axial

link normal to the surface at each nodal point, of stiffness AE/L equal to the tributary soil stiffness. The load is input as concentrated applied forces at the nodal points.

50. For linear tension-compression springs, and linear uncracked concrete structure, the determination of all forces and displacements is a standard problem in the analysis of framed structures which can be solved by either the force or the displacement method.

51. If the springs are active in compression only, and cracking and subsequent stiffness degradation of the structure are considered, then a non-linear problem results which requires an incremental or iterative solution. A program for such an analysis was developed, using an iterative force method approach and containing the stiffnesses described earlier. All further results were obtained using this program.

Applications

Circular Pipe under Cosinusoidal Loading

52. This example is intended to illustrate the effect of the following variables on the soil-structure interaction in a simple fashion:

1. The effect of the soil-structure stiffness ratio on the internal moments.
2. The effect of considering both tensile-compressive, and compression-only, interaction between soil and structure.
3. The effect of structure stiffness degradation due to concrete cracking.

53. The circular pipe shown in Fig. 3-4, of average diameter 120 inches, wall thickness 6", contains inside and outside steel ratio $A_s/bh = .004$, and has a cracking moment M_c in pure bending of 2.96 k-in/in (13.1 KN-m/m).

Bending stiffness varies from that of the uncracked section, EI_g = 68.6×10^3 k-in^2/in (7.75 MN-m^2/m) to that of the cracked section, EI_c = 14.5×10^3 k-in^2/in (1.64 MN-m^2/m) in pure bending. First cracking and subsequent behavior under axial force and moments follows Ref. 3-11 and Fig. 3-5.

54. The soil modulus, which is considered constant all around, ranges in value from zero to 0.2k/in^3 (55.0 MN/m^3), corresponding to a stiffness ratio defined by Eq.2 varying from zero to 35.

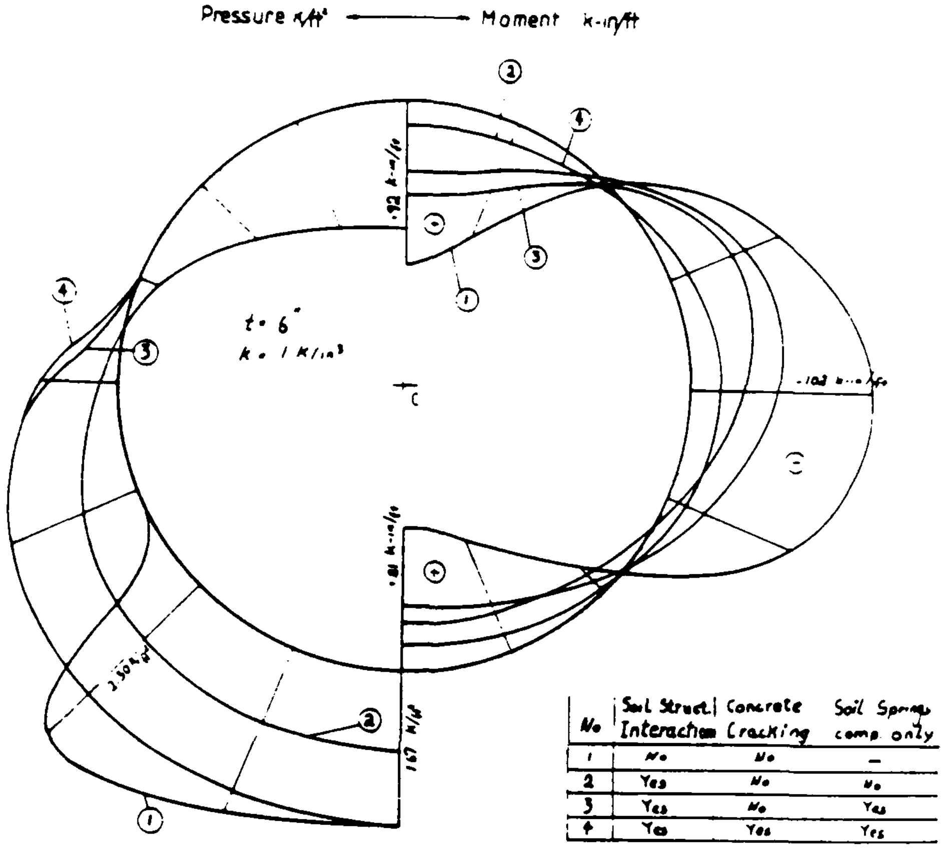
55. The radial load on the pipe varies cosinusoidally over the top 180^o, from a maximum intensity of p at the crown to zero at the springing, representing the radial components of a uniform overburden.

56. We consider four different sets of assumptions in our analyses:

1. No soil-structure interaction. The vertical reaction is supplied by a bed of springs extending over a bedding angle of 90^o .
2. Soil support in tension and compression, with soil stiffness varying from k = zero to 0.20 k/in^3 , corresponding to Υ = 0 to 35.1. Structure stiffness is constant at the uncracked value of EI_g = 68.6 k-in^2/in.
3. Identical to Case 2, but soil reaction is only compressive.
4. Identical to Case 3, but the structure stiffness varies from section to section according to Fig. 3-5, depending on the level of moment and axial compression.

Of these four cases, the first two are linear, and the last two require non-linear analysis.

57. Fig. 3-6 shows the distribution of soil reaction and internal moments throughout the ring according to these four assumptions for the soil stiffness k = 0.10 k/in^3. Moments are largest according to Assumption 1 which



No	Soil Struct. Interaction	Concrete Cracking	Soil Springs comp. only
1	No	No	—
2	Yes	No	No
3	Yes	No	Yes
4	Yes	Yes	Yes

Fig. 3-6. Soil pressure and moments, circular pipe

neglects any lateral soil support. Assumption 2, which considers unrealistic soil tension- compression resistance, results in very low moments, whereas the more realistic Assumption 3 shows moments halfway between. Lastly, the reduced moments based on variable stiffness of the cracked structure, Assumption 4, show the beneficial effects of concrete cracking on the internal forces.

58. Fig. 3-7 focuses our attention on the decrease of the moment M_o at the crown (Nodal Point 0) with increasing soil-structure stiffness ratio, which could be due to either better soil conditions or thinner culvert walls. The vertical axis shows this moment due to Assumptions 2, 3, and 4 as a fraction of that neglecting soil-structure interaction (Assumption 1). The linearization of Assumption 2 is unconservative. But even for the more realistic Assumptions 3 and 4, even a small degree of soil constraint decreases the design moment at the crown by as much as 50 per cent and more. Conversely, for given materials and ring size, a decrease of ring thickness of 25 per cent, from 6 inches to 4.5 inches, can decrease the design moment by a similar percentage. Similar results would be obtained for moments at other sections. The following conclusions can be drawn from these results:

1. Even a small amount of soil constraint serves to decrease bending greatly. For economical pipe design, this should be considered.
2. Reduction of wall thickness will have a similar effect.
3. For realistic results, a non-linear non-tension analysis which considers concrete cracking should be undertaken.

Horseshoe Section with Soil Cover

59. In this example, we will examine the effect of soil-structure interaction on the internal forces and concrete cracking of a horseshoe-shaped

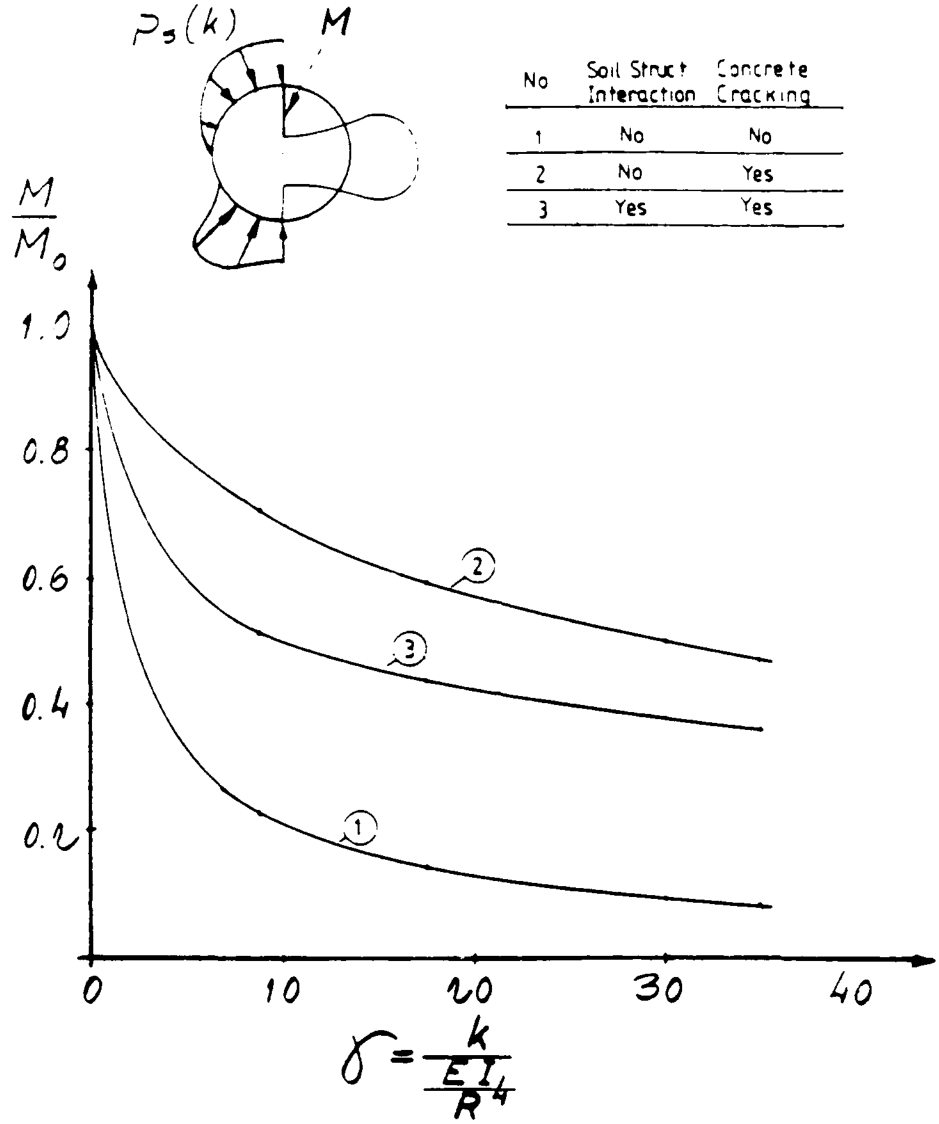


Fig. 3-7. Moment at crown of circular section as function of stiffness ratio

conduit under varying soil cover. The dimensions of the conduit are shown in Fig. 3-8. Its wall thickness is 12 inches, with inside and outside steel ratio of 0.004. Fig. 3-8 also shows the structural discretization for structure and soil, as well as the loads which are due to the overburden of depth C and a linearly increasing lateral load with lateral load coefficient of 0.27, acting over the upper half of the section.

60. In view of the findings of the preceding example, only no-tension springs were used to represent soil pressure. Both uncracked and cracked section behavior was considered. Fig. 3-9 shows the soil pressure and the internal moments due to a soil cover of 15 feet, (1) neglecting soil-structure interaction, (2) considering soil-structure interaction for $k = 0.1\ k/in^3$ and uncracked section, and (3) soil-structure interaction and cracked section. The moments under Assumption 3 are about one third of those under Assumption 1.

61. Fig. 3-10 shows the decrease of the design moment at the crown for the 12 inch thick section with increasing soil stiffness, assuming uncracked and cracked concrete. The trends are quite similar to those observed in the previous example.

62. Eq. 3 shows the strong influence of the wall thickness on the structure response. The section was therefore reanalyzed with the wall thickness reduced from 12 to 6 inches. The results for uncracked section are also plotted in Fig. 3-10 , and show the tremendous moment reduction in the thinner section. In fact, these values indicate that the 6 inch wall will require a smaller reinforcement ratio than the thicker wall.

63. Fig. 3-11 plots the design moment at the crown as function of increasing soil cover, based on different assumptions. Cracking of the concrete under increasing overburden leads to a less than linear increase of

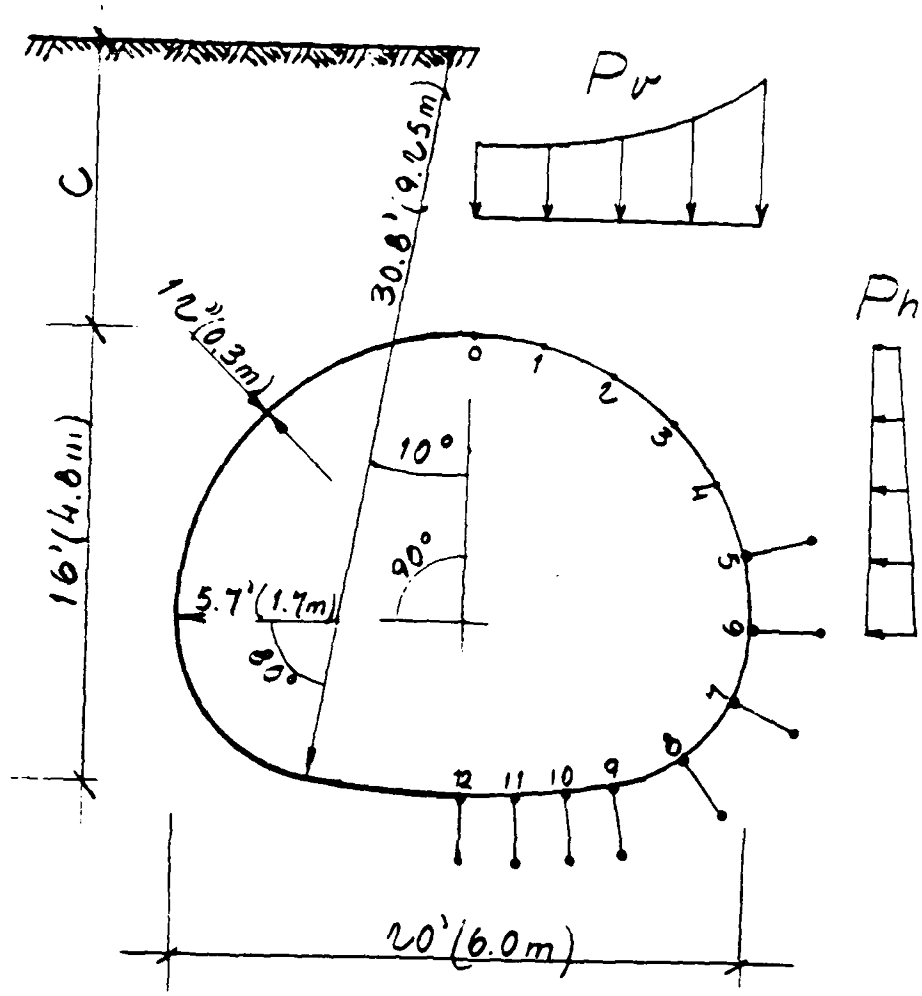
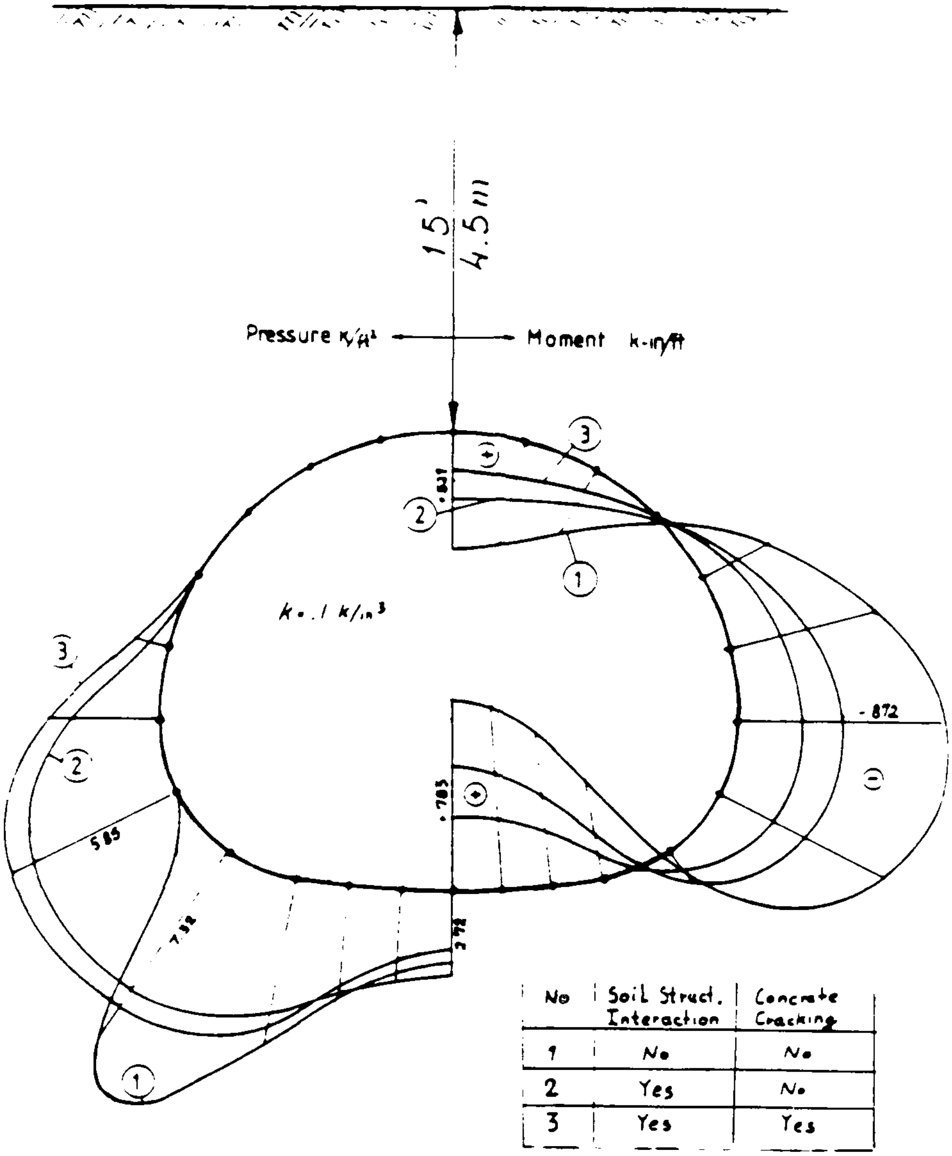


Fig. 3-8. Horseshoe section and structural idealization



No	Soil Struct. Interaction	Concrete Cracking
1	No	No
2	Yes	No
3	Yes	Yes

Fig. 3-9. Soil pressure and moments, horseshoe section

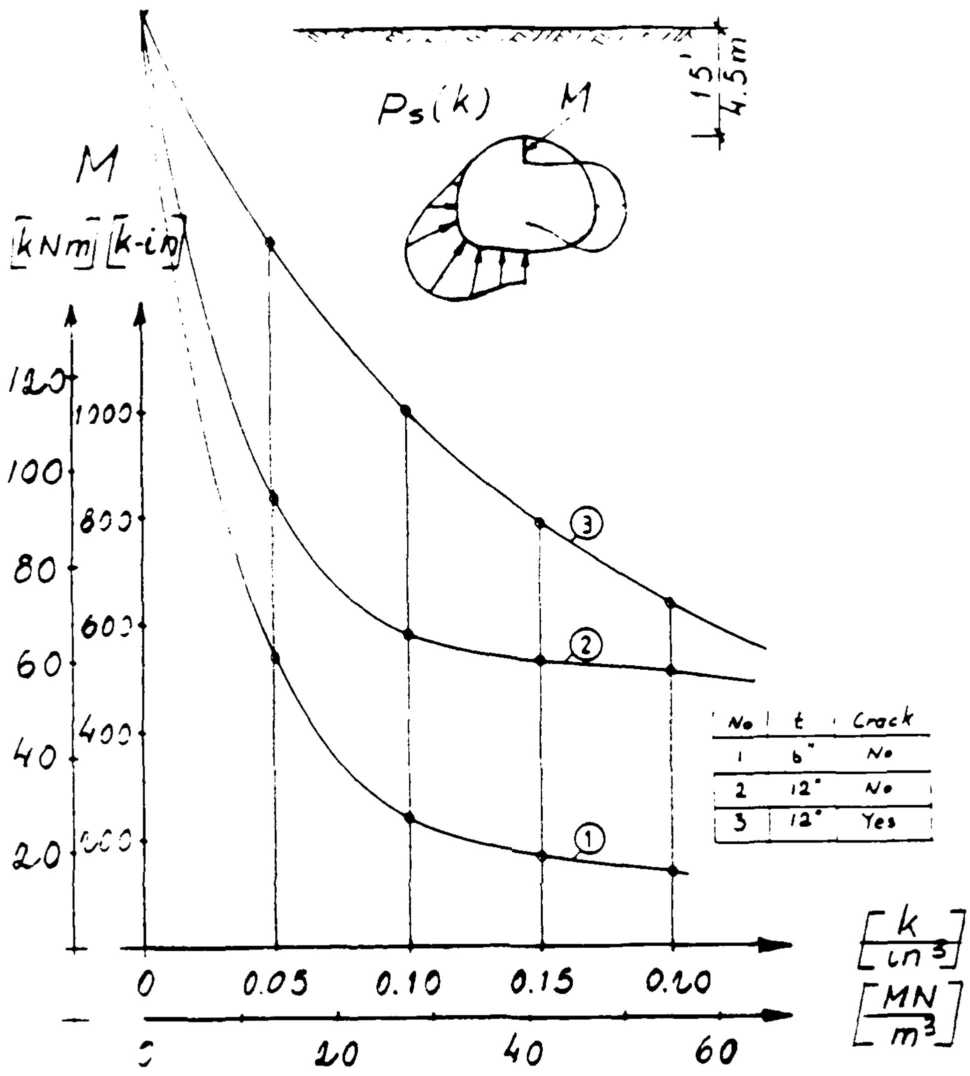


Fig. 3-10. Moment at crown of horseshoe section as function of soil stiffness

No	Soil Struct. Interaction	Concrete Cracking	Arching
1	No	No	No
2	Yes	Yes	No
3	Yes	No	Yes
4	Yes	Yes	No
5	Yes	Yes	Yes

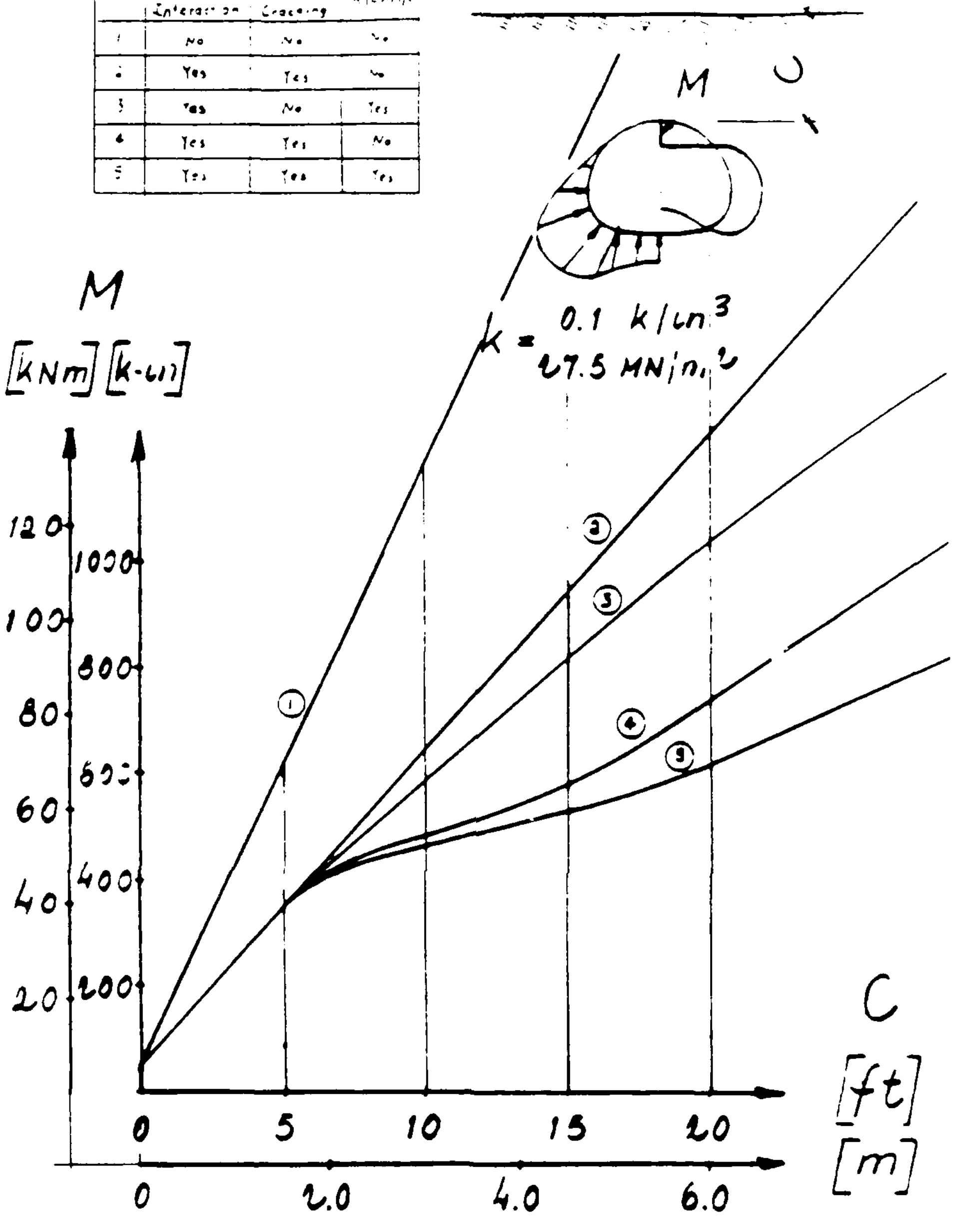


Fig. 3-11. Moment at crown of horseshoe section as function of soil cover

moment, and the arching effect due to Terzaghi (3-13) leads to further moment reduction.

64. An important additional consideration is the concrete cracking which is the cause of the beneficial stiffness reduction of the conduit. Crack widths above 0.01 inches are to be avoided under service conditions.

65. The crack check consists of determination of the internal forces at the critical section, reinforcement design for these factored forces by strength theory, reanalysis and determination of the tensile steel stress. This stress is used in the Lutz-Gergely equation (3-14) to determine the steel arrangement necessary to hold the crack width below the above value.

66. For the critical section at the invert of this conduit, this process led to a maximum bar size of #5 bars at 4 inch spacing on the inside of the wall, with steel stress of 37 ksi under service conditions. This leaves the possibility of a variety of different possible steel arrangements. It appears that crack width will not be a problem. These matters are explored further in Part V. The results of this example confirm the conclusions of the previous analyses.

Preliminary Design by Dimension Ratio

67. Based on field tests carried out in various locations, Bacher (3-4) has suggested incorporating the "dimension ratio" pipe diameter/wall thickness as parameter for culvert pipe design as a means of accounting for the soil-structure interaction. The solid lines of Fig. 3-12, taken from Bacher's work, represent the ratio of soil pressure at the springing line to that at the crown for three different bedding conditions,based on field measurements.

68. The pressure distribution approaches the hydrostatic as the pipe

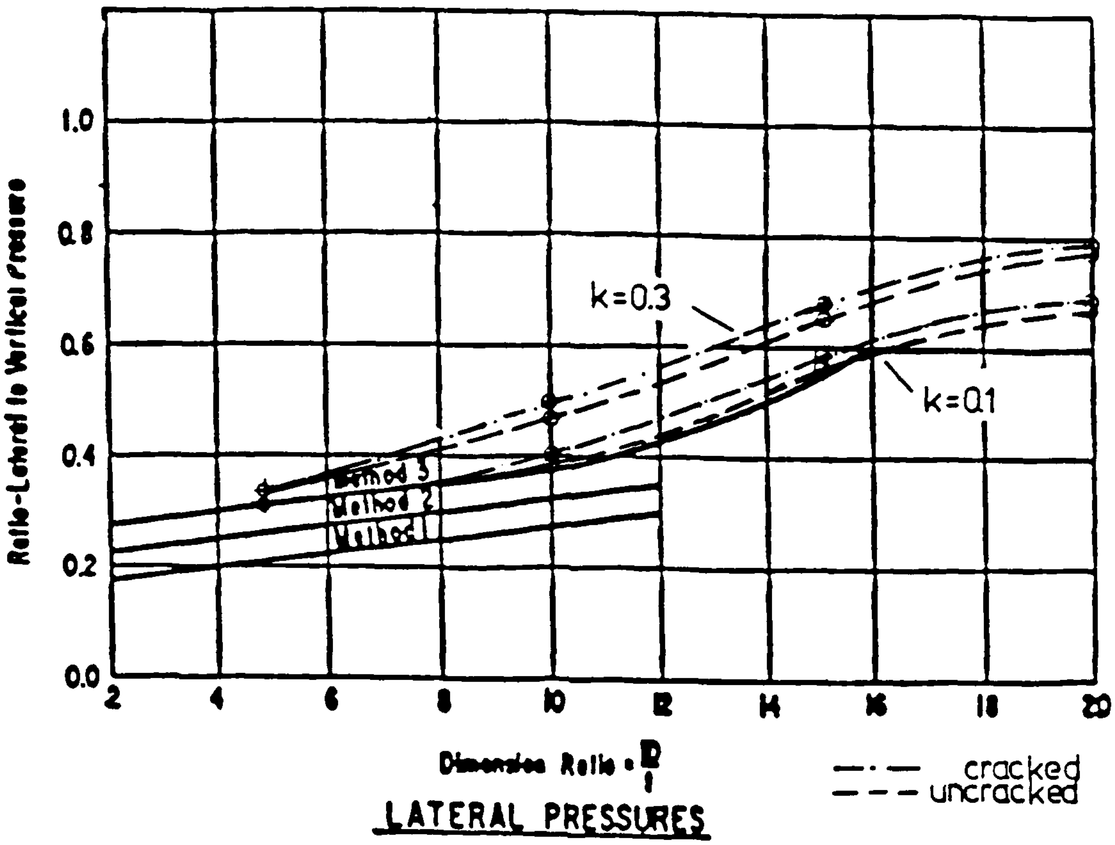


Fig. 3-12 Experimental and analytical soil pressure distribution as function of dimension ratio

wall gets thinner. Based on these observations, Bacher suggested a division of pipes according to their dimension ratio into three classes, "rigid", "semi-rigid", and "flexible", as shown in Fig. 3-13, each class to be designed for the critical soil pressure distributions specified in this figure.

69. We wish to observe whether these ratios can be verified using our simple soil-structure interaction analysis. To this end, circular pipe under the cosinusoidal loading shown in Fig. 3-13 for "rigid" pipe, of lateral to vertical pressure ratio 0.3, was analyzed for interaction with compression-only soil of various stiffnesses. Fig. 3-12 shows the resulting lateral pressure ratios as function of the dimension ratio, superimposed on Bacher's values obtained from field measurements.

70. The solid lines show Bacher's results for various pipe bedding conditions. The dashed lines indicate our analytical results for soil stiffnesses of 0.1 and 0.3 k/in^3, assuming uncracked concrete. The dash-dot lines show similar results based on cracked-section analysis under a soil overburden equal to three pipe diameters. Higher overburdens, producing more concrete cracking, showed only slight differences in pressure distribution. We can draw the following conclusions from a study of Fig. 3-12:

1. The observed trends can be captured by simple soil-structure interaction analysis.
2. The difference between cracked and uncracked section analysis is insignificant.
3. The effect of soil stiffness within the range studied is relatively minor.
4. The lateral force factor suggested by Bacher and shown in Fig. 3-13 for "semi-rigid" and "flexible" pipe may be somewhat on the high side.

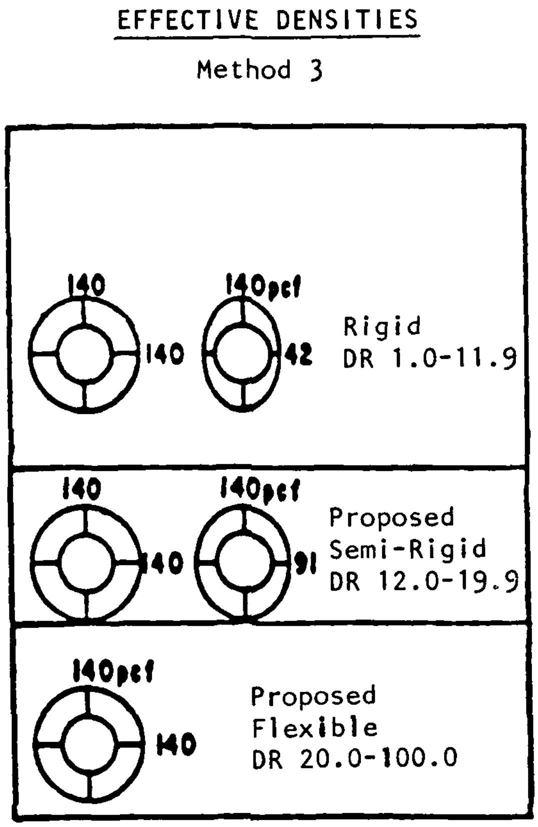


Fig. 3-13 Effect of dimension ratio of circular pipes on soil pressure distribution

71. It appears that rational analysis of soil-structure interaction may be a valuable tool to supplement field observations.

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PART IV: SPECIFICATIONS FOR CONDUIT DESIGN

Introduction

72. In this part, we will outline possible specifications for the analysis and design of reinforced concrete conduits, following these guidelines:

1. Rational analysis for internal forces using accepted principles of structural mechanics and including the effects of soil-structure interaction. The inclusion of the soil as part of the structure is likely to lead to thinner-walled sections than conventional methods which have been used by the Corps of Engineers.
2. Use of results of analytical and experimental soil-structure interaction investigations available in the literature. The simple method proposed in Part III of this report, which models the system as a framed structure, can be used but its final acceptance must depend on comparisons with more refined analyses.
3. Use of the strength method for section and reinforcement design. This method is well known and accepted by designers and much effort has been expended (4-1) to make it applicable to conduit design.

73. While some aspects of this approach are well documented and ready for use, others are at a pioneer or trial stage and will require much more study and documentation before they can be accepted for routine design. For these reasons, in the following some sections will be well documented, while others will be only sketchy. In order to provide guidance for further development, we will proceed in three parts:

1. Suggested specification clauses are designated "S".

2. Commentary on background and questions which may have to be answered in order to attach specific numbers to these clauses, as well as suggestions for design aids are designated "C".

3. Suggestions for specific research needed to provide answers to the outstanding problems are designated "R". Whenever possible we have used the wording of existing specifications for conduit design, since we aim at unification rather than duplication of design methods. Proper references are given in all cases. It is suggested that a committee of experienced design and construction professionals along with appropriate specialists be constituted to insure realism and practicality of these code provisions.

Specifications for Design of Reinforced Concrete Conduits

S 1 Loads, Safety, and Serviceability

S 1.1 Design Loads

74. Design loads consist of the following, to be considered at all critical construction, service, and ultimate stages:

Conduit dead load
Hydrostatic internal and external pressure where applicable.
Soil pressure to be computed on the basis of the soil-structure system.
Surcharge or wheel loads, where applicable.

S 1.2 Safety Factors

75. Ultimate moments, thrusts, and shears required for strength design are determined by multiplying these forces as computed from service load analysis by these safety factors:

Dead load, water pressure, and soil pressure:	1.3
Wheel loads	1.6

S 1.3 Resistance Factors

76. Strength reduction factors shall be applied to the effective section depth as shown in Eqs. 4.1 to 4.10 of Secs. 4.1 to 4.7.

For precast culvert sections, these factors shall be:

For Bending and Axial Compression: $\emptyset = 1.0$
For Shear and Radial Tension: .9
For Cast-In-Place Culverts, these factors shall be:
For Bending and Axial Compression: .9
For Shear and Radial Tension: .85

S 1.4 Serviceability

77. Crack width under critical service conditions shall not exceed 0.01 inch. This requirement is satisfied with reinforcement provided according to Sec. 4.4.

S 2 Analysis

S 2.1 Rational Analysis

78. Internal forces at all sections under critical service conditions shall be determined by rational analysis of soil-structure interaction which considers the ratio of soil to structure stiffness. Such an analysis shall consider non-linearity of soil, if necessary, as well as the effects of cracking of the concrete. Cracking of the concrete can be represented by an equivalent variable stiffness:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

where

$$M_{cr} = \frac{f_r I_g}{y_t}$$

and for normal weight concrete,

$$f_r = 7.5\sqrt{f'_c}$$

79. Determination of the soil response shall follow accepted geotechnical practice. The results of such an analysis shall be moments, axial forces, and shear forces at critical sections, as well as soil pressures.

S 2.2 Simplified Analysis of Circular Pipe

80. In lieu of the rational analysis, the cosinusoidal radial soil pressure distributions shown in Fig. 4-1 may be assumed for circular pipe of different dimension ratio diameter to wall thickness, and bedding conditions designated Methods 1, 2, and 3 and defined in Fig. 4-2.

S 3. Reinforcement Design

81. Reinforcement to resist the factored axial and shear forces and moments from the analysis, as well as to prevent excessive cracking, shall be calculated according to the provisions of Secs. 4.1 to 4.7.

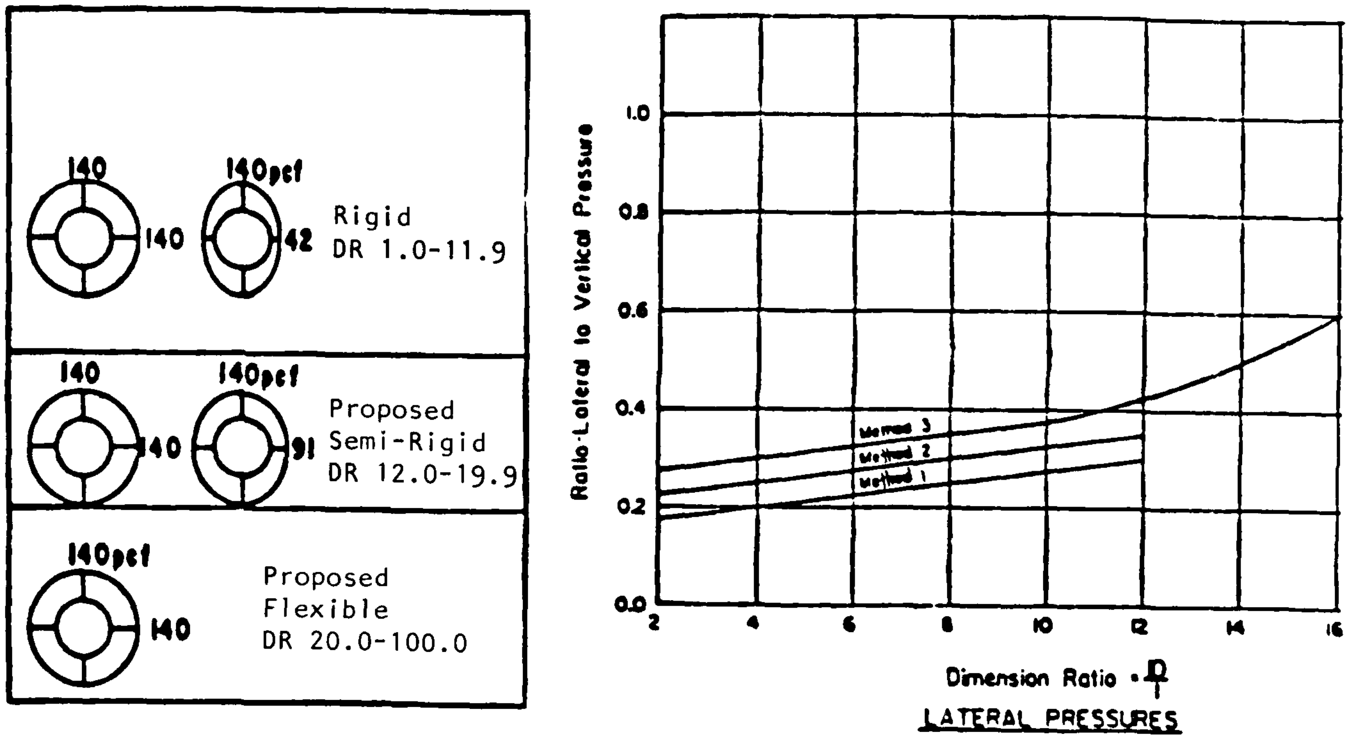


Fig. 4-1. Soil pressure distribution on circular pipe by dimension ratio

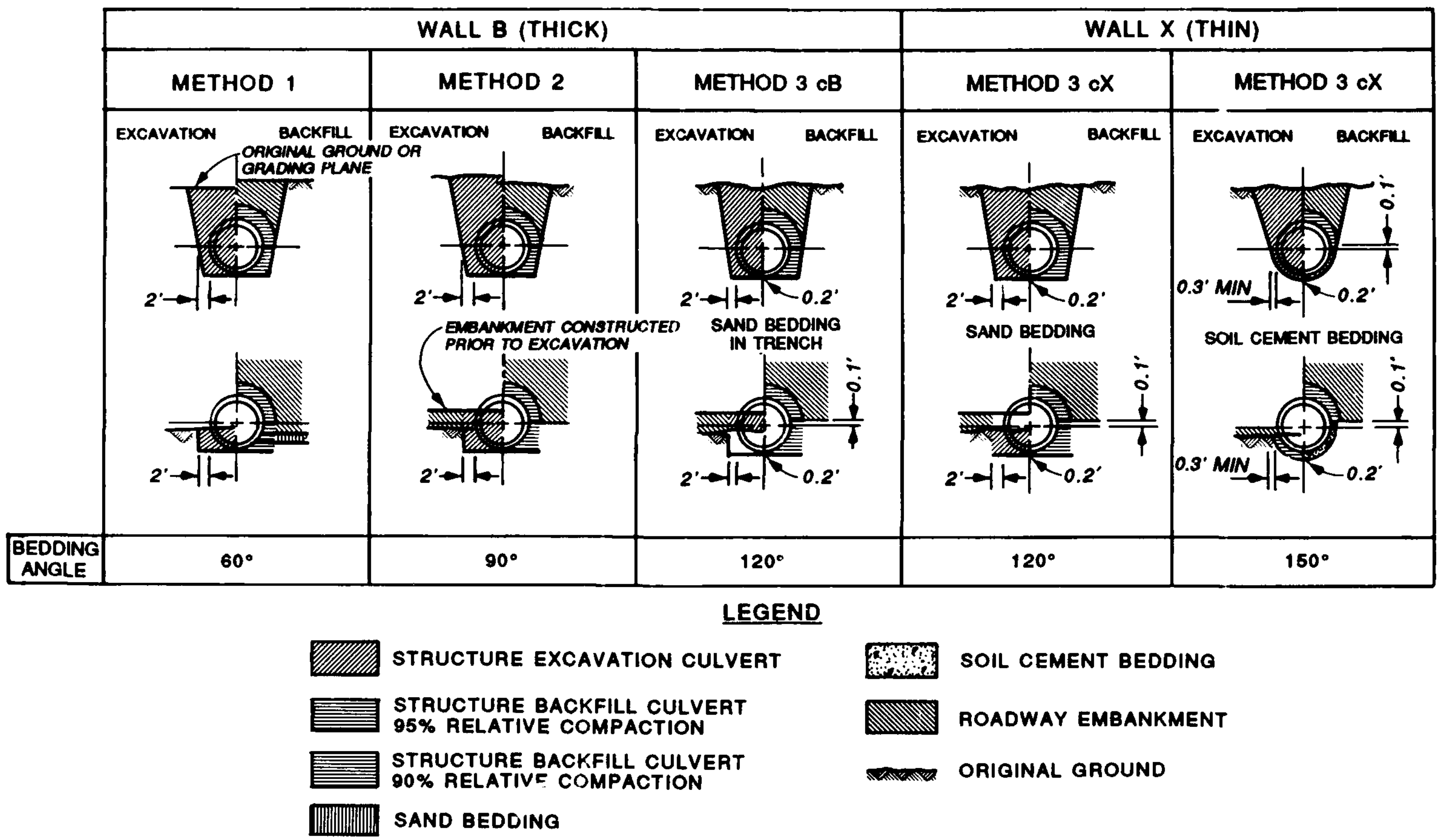


Fig. 4-2. Bedding conditions for circular pipe

Sec. 4.1

Reinforcement for Flexural Strength

$$A_s f_y = g\phi d - N_u - \sqrt{g[g(\phi d)^2 - N_u(2\phi d - h) - 2M_u]} \quad 4.1$$

where $g = 0.85\, bf'_c$

Minimum Reinforcement

For inside face of pipe: $A_s = (S_i + h)^2/65{,}000$ 4.2

For outside face of pipe: $A_s = 0.75\,(S_i + h)^2/65{,}000$ 4.3

For elliptical reinforcement in circular pipe and for pipe 33-inch diameter and smaller with a single cage of reinforcement in the middle third of the pipe wall:

$$A_s = 2\,(S_i + h)^2/65{,}000 \quad 4.4$$

where

h = wall thickness in inches;
S_i = internal diameter or horizontal span of pipe in inches.

In no case shall the minimum reinforcement be less than 0.07 square inches per linear foot.

Sec. 4.2

Maximum Flexural Reinforcement Without Stirrups
Limited by Radial Tension

$$\text{Inside } A_{s\,max}\, f_y = 16 r_s F_{rp} \sqrt{f'_c} \quad 4.5$$

where

$A_{s\,max.}$ = maximum flexural reinforcement area without stirrups in in.^{2}/ft.;
F_{rp} = 1.0 unless a higher value substantiated by test data is approved by the Engineer;
r_s = radius of the inside reinforcement in inches.

Sec. 4.3

Limited by Concrete Compression

$$A_{s\,max.} f_y = \left[\frac{5.5 \times 10^4\, g'\phi d}{(87{,}000 + f_y)}\right] - 0.75 N_u \quad 4.6$$

where

$$g' = bf'_c\left[0.85 - 0.05\,\frac{(f'_c - 4{,}000)}{1{,}000}\right]$$

$g'_{max} = 0.85bf'_c$ and $g'_{min} = 0.65\, bf'_c$

$$F_{cr} = \frac{B_1}{30{,}000\, d A_s}\left[\frac{M_s + N_s\left(d - \frac{h}{2}\right)}{ij} - C_1 b h^2 \sqrt{f_c'}\right] \qquad 4.7$$

where

F_{cr}	= crack control factor, see Note c;
M_s	= bending moment, service load;
N_s	= thrust (positive when compressive), service load;
j	$\cong 0.74 + 0.1\, e/d$;
j_{max}	= 0.9;
i	$= \dfrac{1}{1 - \dfrac{jd}{e}}$
e	$= \dfrac{M}{N} + d - \dfrac{h}{2}$
e/d_{min}	= 1.15;
t_b	= clear cover over reinforcement in inches;
h	= wall thickness of pipe in inches;
B_1 and C_1	= crack control coefficients dependent on type of reinforcement used as follows:

Type Reinforcement:	B_1	C_1
1. Smooth wire or plain bars	$\sqrt[3]{\dfrac{0.5t_b^2 s_\ell}{n}}$	1.0
2. Welded smooth wire fabric, 8 inches maximum spacing of longitudinals	1.0	1.5
3. Welded deformed wire fabric, deformed wire, deformed bars or any reinforcement with stirrups anchored thereto.	$\sqrt[3]{\dfrac{0.5t_b^2 s_\ell}{n}}$	1.9

where

s_ℓ = spacing of circumferential reinforcement in inches;

Notes:

a. Use n = 1 when the inner and the outer cages are each a single layer. Use n = 2 when the inner and the outer cages are each made up from multiple layers.
b. For type 2 reinforcement having $(t_b^2 s_\ell/n > 3.0$, also check F_{cr} using coefficients B_1 and C_1 for type 3 reinforcement, and use larger value for F_{cr}.
c. When $F_{cr} = 1.0$, the reinforcement area, A_s, will produce an average maximum crack width of 0.01 inch. For F_{cr} values less than 1.0, the probability of a 0.01 inch crack is reduced, and for larger values, cracks greater than 0.01 inch may occur.
d. Higher values for C_1 may be used if substantiated by test data and approved by the Engineer.

Sec. 4.5

Shear Strength

The area of reinforcement, A_s, determined in Section 4.1 or 4.4 must be checked for shear strength adequacy, so that the basic shear strength, V_b, is greater than the factored shear force, V_{uc}, at the critical section located where $M_u/V_u \phi d = 3.0$.

$$V_b = b \phi d F_{vp} \sqrt{f'_c}(1.1 + 63\rho)\left[\frac{F_d}{F_c F_N}\right] \qquad 4.8$$

where

V_b = shear strength of section where $M_u/V_u \phi d = 3.0$

F_{vp} = 1.0 unless a higher value substantiated by test data is approved by the Engineer;

$\rho = \dfrac{A_s}{\phi b d} \qquad \rho_{max} = 0.02$

$f'_{c\,max} = 7{,}000$ psi

$F_d = 0.8 + \dfrac{1.6}{\phi d} \qquad F_{d\,max} = 1.25$

$F_c = 1 \pm \dfrac{\phi d}{2r}$ (+) tension on the inside of the pipe
(−) tension on the outside of the pipe

$F_N = 1.0 - 0.12\dfrac{N_u}{V_u} \qquad F_{N\,min} = 0.75$

If V_b is less than V_{uc}, radial stirrups must be provided.

Radial Stirrups

Radial Tension Stirrups

Sec. 4.6

$$A_{vr} = \frac{1.1 s_v (M_u - 0.45\, N_u \phi d)}{f_v r_s \phi d} \qquad 4.9$$

where

A_{vr} = required area of stirrup reinforcement for radial tension;

s_v = circumferential spacing of stirrups ($s_{v\,max} = 0.75\phi d$);

f_v = maximum allowable strength of stirrup material ($f_{max} = f_y$, or anchorage strength, whichever is less).

Sec. 4.7

Shear Stirrups

$$A_{vs} = \frac{1.1 s_v}{f_v \phi d}\left[V_u F_c - \phi V_c\right] + A_{vr} \qquad 4.10$$

where

A_{vs} = required area of stirrups for shear reinforcement;

V_u = factored shear force at section;

$V_c = \dfrac{4V_B}{\dfrac{M_u}{V_u \phi d} + 1}$

$V_{c\,max} = 2\phi b d \sqrt{f'_c}$

Commentary on Conduit Specifications

C 1.1 Design Loads

82. Specified design loads due to earth pressure may be those acting on rigid conduit structures. These soil pressures have been specified in many regulations, for instance:

1. Corps of Engineers, EM 1110-2-2902, 1969, Sec. 4
2. AASHTO Highway Bridge Specifications, 1988 Sec. 6.2, Sec. 6.4, Sec. 17.4.

These soil loads may be considered acting on rigid culverts since they do not depend on the soil-structure stiffness ratio. The soil-structure analysis specified in Sec. S 2.1 will modify these pressure distributions on the basis of the deformation of the flexible conduit section.

83. This approach calls for the choice of culvert cross sections so they will act as shells able to resist soil pressure as much as possible by membrane action, such as circular, elliptic, or arch-type structures. Conduits with straight wall segments will not be optimal under this system.

C 1.2 Safety Factors

84. The safety factor is applied to the internal forces at service level, rather than to the service loads, because soil-structure interaction will result in non-linear relations between applied loads and internal forces, as shown in Fig. 4-3. This figure shows that non-linear action leads to variable safety at different sections for one specified load factor. The safety factor of 1.3 has been suggested by the Technical Committee of ACPA for analysis of rigid conduits under conservative assumptions (4-2). Other specifications are more conservative, such as ACI 318-83 Sec. 9.2 (4-3), AASHTO 1988 Sec. 3.22 (4-4).

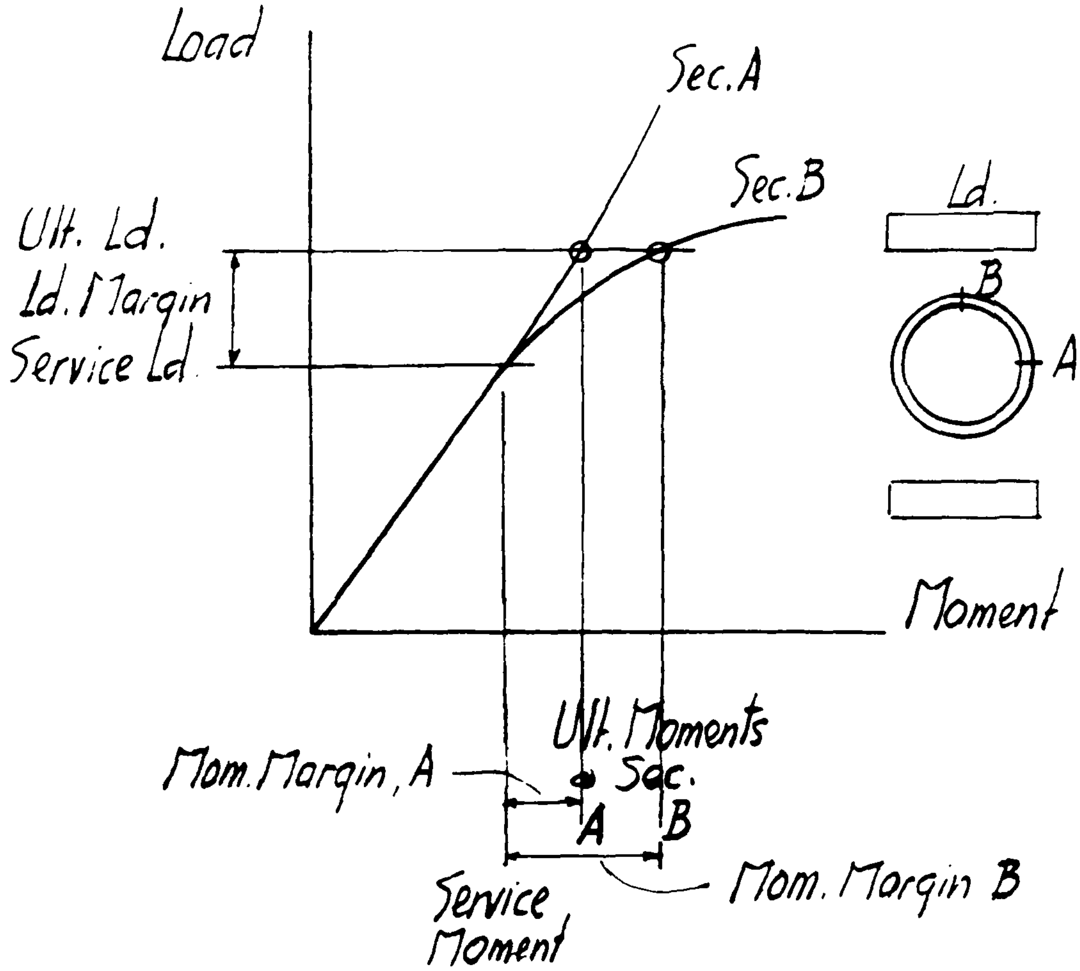


Fig. 4-3. Non-linear load-moment relations

C 1.3 Resistance Factors

85. The resistance factors specified in Sec. S 1.3 for precast pipe follow those of Sec. 17.4.6.2 of the 1984 AASHTO Specifications (4-4). Those for cast-in-place construction are from ACI 318-83, Sec. 9.3, with the exception of that for combined axial load and bending. In conduits, bending will usually predominate, resulting in tension failure of the section. The resistance factor for radial tension is the same as for shear, since both phenomena depend on the highly random tensile strength of concrete.

86. Heger pointed out (4-2) that the principal uncertainty regarding section strength is due to possible misplacement of the steel. For this reason, he suggested attaching the resistance factor to the effective section depth, as shown in Eqs. 4.1 to 4.10.

C 1.4 Serviceability

87. Critical service conditions are those leading to maximum tensile steel stress. The allowable crack width of 0.01 inch is in conformity with design practice for hydraulic structures, although it has been claimed (4-5) that cracks as wide as 0.1 inch will not lead to either corrosion or safety problems.

88. Cracking in flexible culverts is necessary to achieve the beneficial passive soil pressures leading to moment redistribution. The circumferential reinforcement should be arranged so that the cracks will be small and well distributed, as provided for in Sec. S 3.4.

C 2.1 Rational Analysis

89. The most accurate, but also most demanding, type of analysis is a non-linear finite-element solution of the complete soil-structure system, such

as the Program SPIDA (4-6). Such a program can predict conditions at all stages of construction and service, given sufficient input information. Simpler analyses can use readily available computer programs for analysis of framed structures, in which the soil constraints can be represented by radial or tangential struts of appropriate stiffness. Such in-house programs are usually linearly-elastic, but by suitable nesting or sequential analyses can be used iteratively for non-linear analyses. Such a solution is used in Part III of this report.

90. The inclusion of concrete cracking and the consequent stiffness degradation is necessary to capture the pressure redistribution due to conduit deformations. This stiffness reduction as represented in Sec. S 2.1 is taken from ACI 318-83, Sec. 9.5 (4-3). It does not include the effect of axial compression and may overestimate the increase in flexibility. A more rational approach to the prediction of stiffness degradation which includes the effects of axial compression as well as tension stiffening is presented in Ref. 4-7. It could be used as an alternate to the simpler ACI method, as was done in the analysis of Part III.

91. The appropriate description of soil behavior should be obtained from a geotechnical specialist. Soil pressures are specified as part of the solution to permit a check on possible soil failure.

C 2.2 Simplified Analysis of Circular Pipe.

92. These provisions follow recommendations to the California Department of Transportation (4-8, 4-9) based on field measurements of full-scale experimental pipes. They are offered here for discussion because they suggest an attractively simple method of accounting for soil-structure interaction for

circular pipe. Analyses in Part III of this report provide partial analytical corroboration of these results. Much more experimental and analytical work will be necessary before these guidelines can be accepted for actual conduit design.

C 3 Reinforcement Design

93. These specifications are taken in their entirety from the 1984 AASHTO Specifications for Highway Bridges, Chapter 17 (4-4), which appears to represent the state of the art in culvert design. These provisions are largely based on the work of Heger and are well documented in Refs. 4-1, 4-10, 4-11, and 4-12.

C3.1 Flexural Reinforcement

94. Eq.4.1 follows standard strength theory for concrete sections reinforced on the tension side only. For symmetrically reinforced sections, an alternate design procedure would be the use of strength interaction curves such as those in Ref. 4-13. For unsymmetrically reinforced sections, appropriate strength interaction curves could be constructed. In fact, Heger suggests optimal reinforcing for conduit pipes consisting of exterior steel equal to 60 per cent of the interior steel (4-14).

95. The minimum steel requirements of Eqs. 4.2, 4.3, and 4.4 depend on the wall thickness and dimension ratio, as shown in Fig. 4-4, which demonstrates that these requirements are in general well in excess of the minimum steel ratio of 0.002 recommended by Heger (4-1).

C 3.2 and 3.6 Radial Tension

96. Slabbing at the inside of curved sections due to radial tension

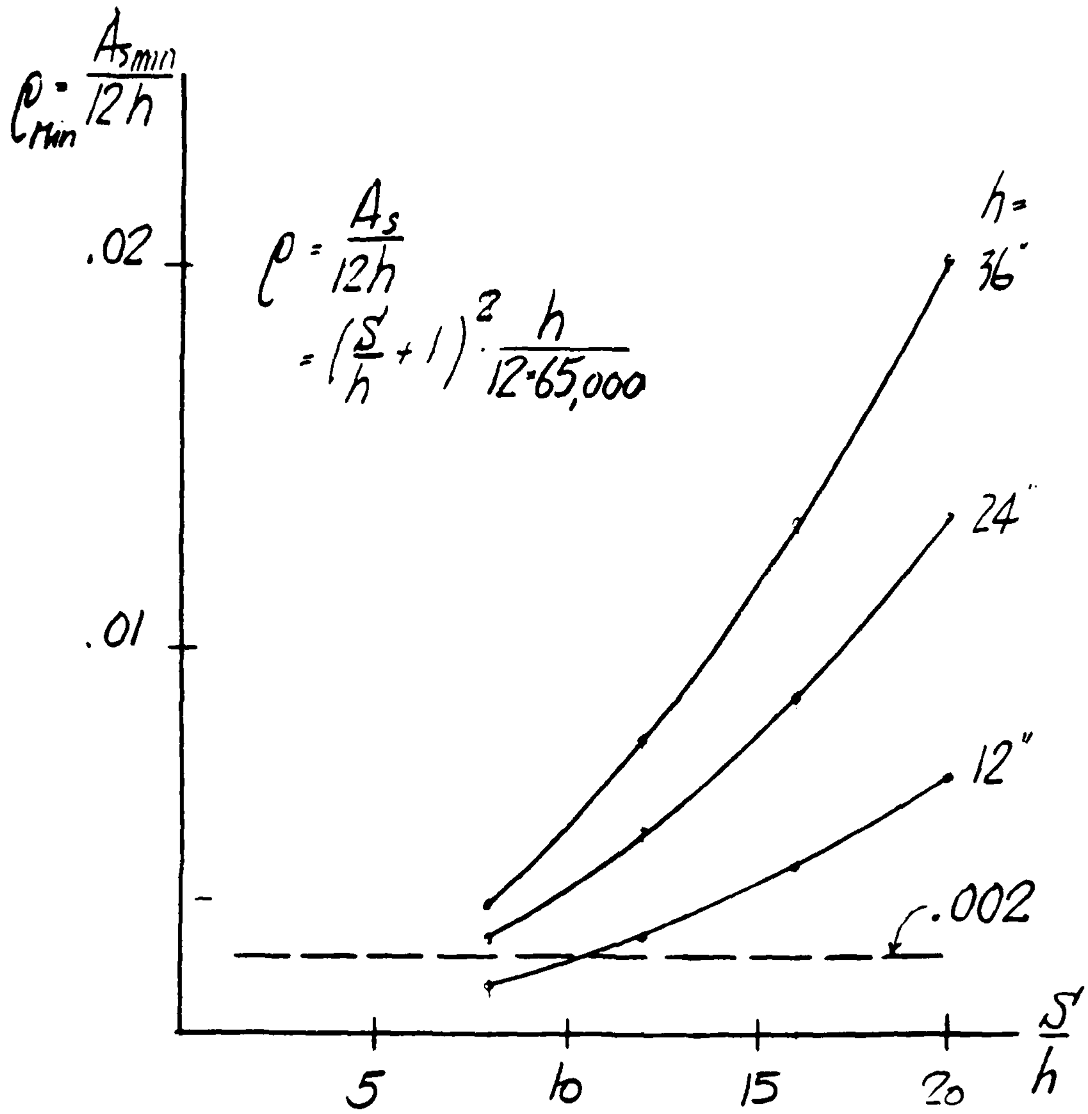


Fig. 4-4. Minimum flexural steel requirements according to Eq. 4.2

stresses will not be a problem unless the inside steel exceeds the value specified by Eq. 4.6. Otherwise, radial stirrups will be required according to Eq. 4.9.

C 3.3 Compression Failure

97. Eq. 4.1 presumes tensile yielding of the steel prior to compressive crushing. To insure such tension failure (which will usually be the case in conduit walls), Eq. 4.6 prescribes maximum reinforcement.

C 3.4 Crack Width Control

98. Crack control according to Eq. 4.7 is a modification of the Lutz-Gergely equation (4-15). This simpler expression lends itself to plots such as the one shown in Fig. 4-5. It clearly shows the importance of the tensile steel stress under working conditions. The choice of allowable crack width of 0.01 inch was discussed in Ref. 4-5, where it is claimed that wider cracks may be allowable under certain conditions. Equation 4.7 appears to have been developed for computer implementation, and as such its physical meaning is obscure. In fact, the first term denotes the applied moment about the tension steel, and the second term the cracking moment, so that the term within brackets represents the increment of moment beyond the cracking value. Thus, a negative value indicates no cracking, a case which is obscured by the probabilistic approach to this problem.

C 3.5 and 3.7 Shear Strength

99. The determination of the basic diagonal tension strength of the section by Eq. 4.8, and the provision of shear stirrups according to Eq. 4.10, follows ACI theory as modified by Heger (4-10). This approach may lead to

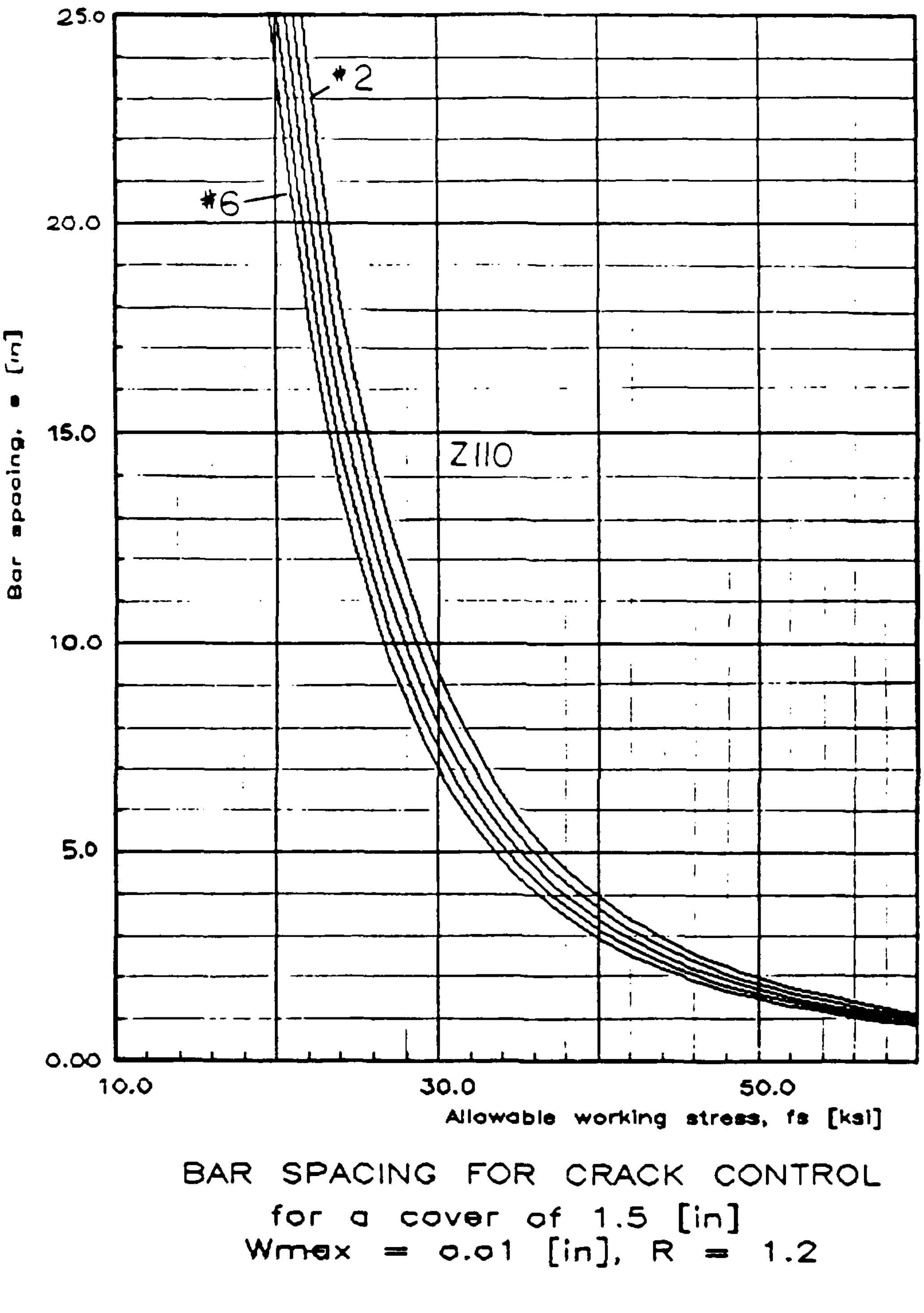


Fig. 4-5. Crack control according to Lutz and Gergely

conservative results as discussed in Ref. 4-16.

100. The definition of V_{uc} in Sec. 4.5 as the factored shear force at the critical section defined by $M_u/V_u\phi d = 3.0$ was derived from shear tests on beams under concentrated loads, and is inappropriate for the type of shear and moment distribution found in culvert sections under soil loading.

Research Notes on Conduit Specifications

101. Much additional work is necessary before these specifications can be implemented with confidence. In the following, some points are raised which need to be explored. These should not be considered exclusive.

R 1 Safety and Serviceability

102. R 1.1 Soil pressures on rigid structures are a recurrent topic in the literature. Appropriate choices for specifications and design procedures should be made by geotechnical experts.

102. R 1.2 Safety factors, and choice of load versus strength factors and their effects on serviceability and safety of the resulting structure require input from specialists on structural safety and probability.

103. R 1.4 There is considerable diversity of opinion on permissible crack width in a non-corrosive environment and much more information on the performance and durability of cracked conduits in service is needed. The Lutz-Gergely approach (4-15) to crack width determination is widely known and has been adopted by ACI. Whether the refinements of Eq. 4.7 are needed should be checked. The basis of all of these approaches is purely empirical. European approaches (4-7) to crack width control are more rational and more complex. Comparison between these different predictions and observed cracking

would be useful so that a method combining reality with simplicity can be specified for use.

R 2.1 Rational Analysis

104. "Exact" analysis programs, such as SPIDA (4-6) and CANDE (4-17), should be checked for suitability for office practice, and the results should be correlated for reliability and ease of interpretation. Since such programs may be too demanding for routine office use by designers, the possibility of compiling "exact" results in tabulated or graphical non-dimensional form for various common conduit shapes and bedding and soil conditions should be explored.

105. It should be clear that if the soil is used as part of the structure, greater control of soil placement and compaction during construction will be necessary and must be clearly specified. Contractors and field engineers should be consulted regarding feasibility and economy of such requirements.

106. Appropriate modeling of the soil-structure system as framed structure must be explored by comparison with "exact" solutions and with field measurements. The following aspects in particular should be checked:

1. Modeling of culvert sections by straight beam elements, and necessary degree of discretization.
2. Representation of concrete stiffness degradation due to cracking.
3. Representation of soil constraint by axial struts. Need for tangential struts to model surface friction. Compression-tension behavior; need to consider non-linear soil behavior.
4. Modeling of the soil loads on the conduit.

The development of simple, reliable soil-structure analysis methods appears to be one of the potentially most rewarding areas of activity.

R 2.2 Simplified Analysis

107. The approach of this section, as proposed in Refs. 4-8 and 4-9, is based on scant field measurements and the conclusions of these references have been strongly attacked (4-18). Extensive field, laboratory, and analytical work will be necessary prior to any adoption. The "Dimension Ratio" example of Part III of this report indicates the possible sound analytical basis for the approach, which appears sufficiently attractive to warrant further effort.

R 3 Reinforcement Design

108. As discussed in the Commentary, Sec. C 3.1, design aids or procedures which consider doubly-reinforced sections under axial compression and moment should be developed to replace Eq. 4.1 which considers only steel on the tension side. As discussed in Ref. 4-16, the theory for shear failure of culvert sections which underlies Eqs. 4.8 and 4.10 may be based on questionable assumptions, and deserves a thorough analytical and experimental study. Similarly, Eq. 4.7 for crack control needs further study, as already pointed out in Sec. C 3.4.

Computer Implementation of Reinforcement Design

109. A computer program was written for reinforcement design, which, when used together with the analysis program discussed in Part III, constitutes a powerful tool for efficient conduit design. In this section, use of this technique is described and demonstrated by means of an example design. The design program, written in FORTRAN 77, is listed in Appendix A.

110. The design program consists of three parts: Design for flexural strength, Eqs. 4.1 to 4.4, and 4.6 of the Specifications, crack control according to Eq. 4.7, and design for radial tension according to Eqs. 4.5 and 4.9, and for diagonal tension according to Eqs. 4.8 and 4.10. The results are output in convenient format as illustrated below. This program was written in order to carry out example designs within the scope of this report, and does not represent a finished design tool for office use. Additional effort will be required to make this a foolproof, user-friendly design tool.

111. This program must be used in an iterative fashion with the analysis program: with given cross-sectional dimensions and wall thickness, and an assumed steel ratio, a soil-structure interaction analysis is performed for the specified soil loads and soil stiffness, as in Part III. The resulting internal forces are used as input in the design program. For design, critical sections are identified by inspection of the analysis results, and steel requirements are determined at these sections so that Eqs.4.1 to 4.10 of the Specifications are satisfied.

112. If the flexural steel required by this design is different from the initially assumed value, a new analysis is carried out, and the design is repeated to convergence. In the design examples of this and the following Part V, the process converged sufficiently fast so that the steel selected in the first iteration did not need modification. The calculation of the flexural steel area required according to Eqs. 4.1 to 4.4, and 4.6, is straightforward. If the minimum steel according to Eq. 4.1 exceeds the maximum steel allowed by Eq.4.6, a prompt will ask for a thicker wall.

113. Crack control according to Eq. 4.7 is somewhat more complex. If according to the current analysis iteration a section is uncracked, no further

calculations are carried out. Otherwise, a value for F_{cr} corresponding to the permissible crack width specified in the input is inserted and Eq. 4.7 is solved for B1. For reinforcement Types 1 and 3 as defined in Sec. 4.4, the maximum allowable spacing s is extracted from the definition of B1. In the case of reinforcement Type 2, the minimum value of A_s is determined directly from Eq. 4.7.

114. For radial tension, Eq. 4.5 determines the need for stirrups; if the required flexural steel area is below the value specified by this equation, no further computation is necessary. If above, the program advances to Eq. 4.9 to determine the stirrup spacing A_{vs}/s.

115. For diagonal tension, the critical section as defined in Sec. 4.5 is ignored, as discussed earlier in the Commentary. Rather, the factored shear force at all sections considered is compared to the basic shear strength V_b as determined by Eq. 4.8; if it is below this value, no further calculation is needed; if above, the required stirrup area at this section, A_{vs}/s, is determined by Eq. 4.10. The input and output format is described in the following example design problem.

Design Example

116. The procedure is demonstrated for the design of the reinforcing of the conduit section shown in Fig. 4-6, subject to the uniform loads also shown in the figure. No soil-structure interaction is considered here. The proportions and loads of this structure were chosen so as to illustrate all the features of the design program. Further discussion will refer to Table 4- A.

117. The cross-sectional properties which were input are shown in the input echo labeled "Cross Section" in Table 4-A, load and resistance factors

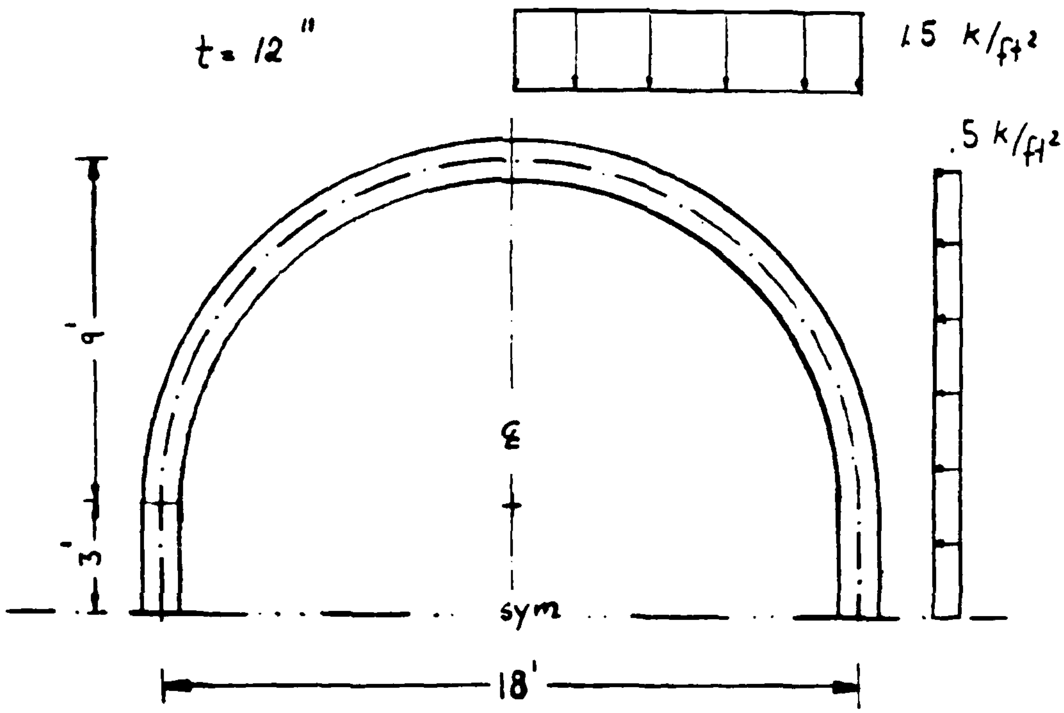


Fig. 4-6. Design example 4.1: Oblong section conduit

* P R O G R A M   P I P E   D E S I G N *

Design Example 4.1 Oblong Pipe with const. Pressure, no Soil-Springs

CROSS SECTION

Pipe Width	s =	216.00	[in]
Segment Length	b =	12.00	[in]
Thickness	t =	12.00	[in]
Cross Section	Ab =	144.00	[in2]
Steel Ratio	roh =	0.40	[%]
Clear Cover	tb =	1.00	[in]
CL Steel Cover	dc =	1.50	[in]
all. Crack Width	crw =	0.010	[in]

SAFETY FACTORS

Reduction Factor	phif =	0.90	[/]
	phiv =	0.85	[/]
Load Factor	rlo =	1.30	[/]

MATERIAL PROPERTIES

Compression Strength	fc =	4.00	[ksi]
Yield Strength Steel	fy =	60.00	[ksi]

>D E S I G N - R E S U L T S<

Table 1 : Section Features

1 Section No	2 Mu [k-in/ft]	3 Nu [kips/ft]	4 Vu [kips/ft]	5 req. Steel As [in2/ft]	6 Ratio roh [%]
0	+957.540	-7.804	+0.000	+1.939	+1.347
1	+837.598	-10.092	+8.281	+1.619	+1.124
2	+511.343	-16.328	+14.242	+0.822	+0.571
3	+70.394	-24.804	+16.193	+0.800	+0.555
6	-729.621	-40.995	-1.947	+1.101	+0.764

Table 2 Reinforcement for Flexural Strength

1 Section No	2 As [in2/ft]	3 Asmin Inside	4 Asmin Outside	5 Asmax Compres
0	1.939	0.800	0.600	2.307
1	1.619	0.800	0.600	2.278
2	0.822	0.800	0.600	2.200
3	0.000	0.800	0.600	2.094
6	1.101	0.600	0.800	1.892

Table 3 : Crack Control Design

1 Section No.	2 max. spacing [in] Type 1	3 Type of Reinforcement Type 3	4 As [in2/ft] Type 2
0	1.48	2.36	1.973
1	1.48	2.63	1.617
2	2.01	8.43	0.648
3	0.00	0.00	0.000
6	2.10	5.84	0.909

Type 1 : Smooth Wire or Plain Bars
Type 2 : Welded Smooth Wire Fabric
Type 3 : Welded Deformed Wire Fabric

Table 4 : Radial and Diagonal Tension Reinforcement

1 Section No.	2 Asmaxr [in2/ft]	3 Vb [kips/ft]	4 Vc [kips/ft]	5 Avr/s [in/ft]	6 Avs/s [in/ft]
0	1.746	24.237	13.547	0.018	0.018
1	1.746	19.501	6.325	0.000	0.000
2	1.746	14.896	11.863	0.000	0.000
3	1.746	15.609	13.547	0.000	0.011
6	1.746	20.630	1.920	0.000	0.000

Table 4-A. Preliminary design, assumed steel ratio = .004

* P R O G R A M   P I P E - D E S I G N *

Design Example 4.1 Oblong Pipe with const. Pressure, no Soil Springs

CROSS SECTION

```
Pipe Width          s  =   216.00 [in]
Segment Length      b  =    12.00 [in]
Thickness           t  =    12.00 [in]
Cross Section      Ab  =   144.00 [in2]

Steel Ratio       roh  =     1.35 [%]
Clear Cover        tb  =     1.00 [in]
Cl Steel Cover     dc  =     1.50 [in]
all. Crack Width  crw  =    0.010 [in]
```

SAFETY FACTORS

```
Reduction Factor phif  =     0.90 [/]
                 phiv  =     0.85 [/]
Load Factor       clo  =     1.30 [/]
```

MATERIAL PROPERTIES

```
Compression Strength  fc  =   4.00 [ksi]
Yield Strength Steel  fy  =  60.00 [ksi]
```

\> D E S I G N - R E S U L T S <

Table 1 : Section Features

1 Section No.	2 Mu [k-in/ft]	3 Nu [kips/ft]	4 Vu [kips/ft]	5 req. Steel As [in2/ft]	6 Ratio roh [%]
0	+973.349	-7.808	+0.000	+1.981	+1.376
1	+853.421	-10.096	+8.281	+1.658	+1.151
2	+52[illegible].207	-16.332	+14.240	+0.856	+0.594
3	+86.321	-24.807	+16.190	+0.800	+0.555
6	-713.408	-40.995	-1.951	+1.061	+0.737

Table 2 : Reinforcement for Flexural Strength

1 Section No.	2 As [in2/ft]	3 Asmin Inside	4 Asmin Outside	5 Asmaxc Compres
0	1.981	0.800	0.600	2.307
1	1.658	0.800	0.600	2.278
2	0.856	0.800	0.600	2.200
3	0.000	0.800	0.600	2.094
6	1.061	0.600	0.800	1.892

Table 3 : Crack Control Design

1 Section No.	2 Type of Reinforcement max. spacing [in] Type 1	3 Type 3	4 As [in2/ft] Type 2
0	1.48	2.35	2.016
1	1.48	2.59	1.660
2	1.94	7.46	0.691
3	0.00	0.00	0.000
6	2.14	6.25	0.865

Type 1 : Smooth Wire or Plain Bars
Type 2 : Welded Smooth Wire Fabric
Type 3 : Welded Deformed Wire Fabric

Table 4 : Radial and Diagonal Tension Reinforcement

1 Section No.	2 Asmaxr [in2/ft]	3 Vb [kips/ft]	4 Vc [kips/ft]	5 Avr/s [in/ft]	6 Avs/s [in/ft]
0	1.746	24.500	13.547	0.019	0.019
1	1.746	19.719	6.286	0.000	0.000
2	1.746	15.084	11.720	0.000	0.000
3	1.746	15.610	13.547	0.000	0.011
6	1.746	20.356	1.940	0.000	0.000

Table 4-B. First design iteration, steel ratio = .0135

are shown in the echo labeled "Safety Factors". Material properties are shown under the appropriate title. From the results of the first analysis, performed for the section with assumed steel ratio of 0.4 per cent, critical sections 0,1,2,3, and 6 are identified, and the factored internal forces at these sections are printed out in Columns 2 to 4 of Table 1 of Table 4-A. The required steel areas and ratios in Columns 5 and 6 are a summary of the design results itemized later, printed here for convenient comparison with the initially assumed steel ratio. If this steel ratio is sufficiently close to that assumed initially, no further iteration is needed.

118. In the present case, steel ratios ranging from 0.55 to 1.35 per cent are shown, as compared to the initially assumed one of 0.4 per cent, so that another iteration is indicated. Thus, a new steel ratio of 1.35 per cent, as shown under "Cross Section" in Table 4-B, was assumed, and a new analysis was carried out, leading to the results displayed in Table 1 of Table 4-B. In particular, Column 6 shows maximum inside steel ratio of 1.38 at Section 0, outside steel ratio of 0.55 at Section 3, indicating near-convergence to the assumed value. Further discussion will refer to the design results displayed in Table 4-B. Table 2 of Table 4-B shows the flexural strength checks according to Eqs. 4.1, 4.2, 4.3, and 4.5 in Columns 2 to 5 for the critical sections. Column 5 shows that compression failure is of no concern in this case. The maximum value of Columns 2, 3, and 4 is the one listed in Column 5 of Table 1 of Table 4-B.

119. Table 2 of Table 4-B shows the results of Eq. 4.7 for crack control. The non-zero values for all sections except No. 3 indicate cracking. At these sections, the maximum permissible bar spacing for reinforcing Types 1 and 3 is shown in Columns 2 and 3, and the minimum steel area for Type 2 in

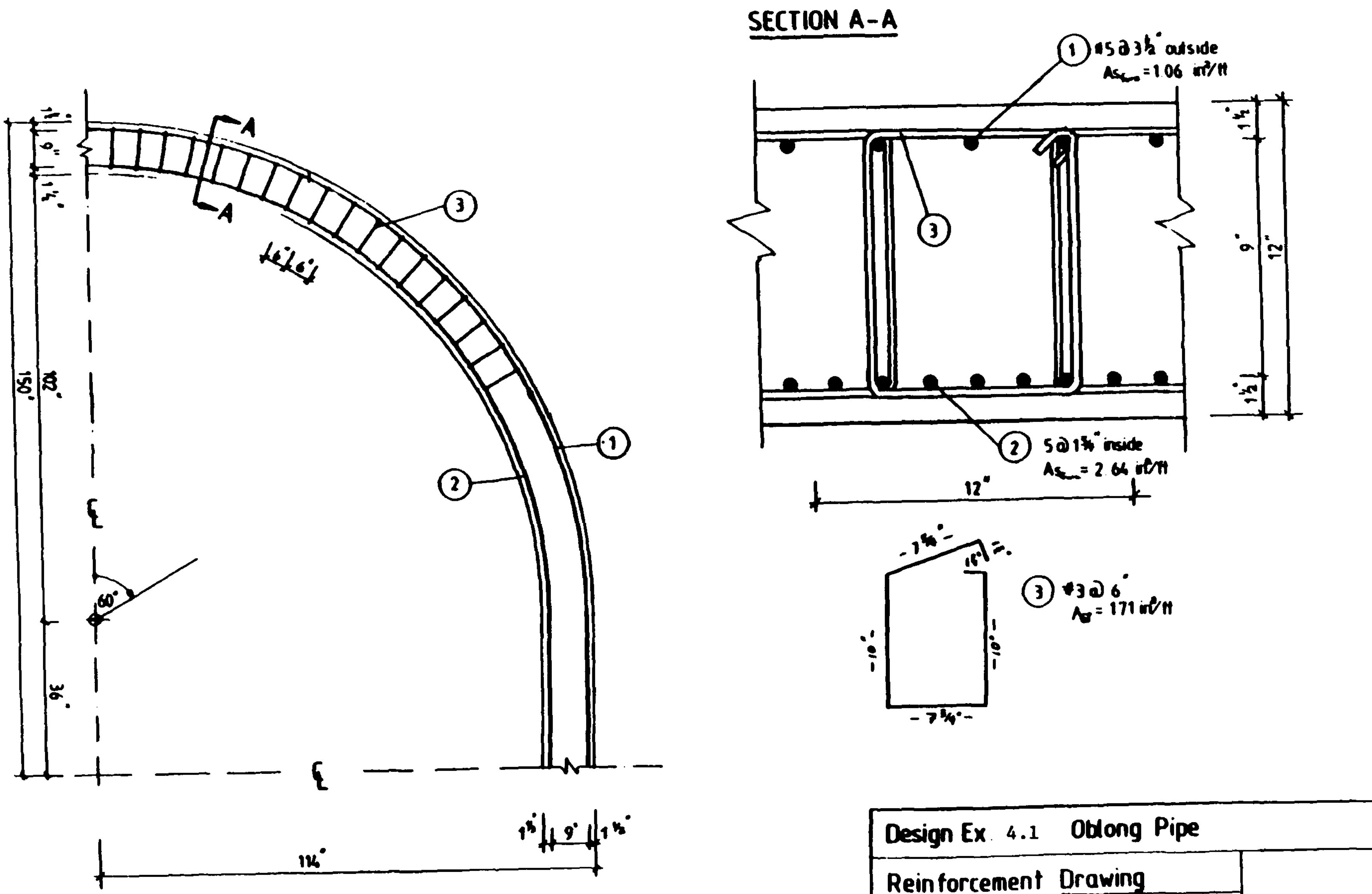
Column 4. The very tight bar spacing of 1 1/2 inches at the crown may be very hard to implement for Type 1 reinforcement.

120. Table 4 of Table 4-B shows the results of the radial and diagonal tension checks. Column 2 shows the maximum steel area for radially unreinforced sections according to Eq. 4.5. This value is exceeded by the required flexural steel only at the crown Section 0, and Column 5 shows the required stirrup spacing at this section. The zeroes at all other sections under Column 5 indicate that no radial tension stirrups are needed there.

121. The basic shear strength at all critical sections according to Eq. 4.8 is displayed in Column 3. Only at Section 3 does the factored shear force shown in Column 4 of Table 1 of Table 4-B exceed this value V_b, and only at this point are diagonal tension stirrups needed. At this section, the required diagonal tension stirrup area is computed by Eq. 4.10, for which the needed value of V_c according to Sec. 4.7 of the Specifications is shown in Column 4 of Table 4 of Table 4-B.

122. Column 6, finally, shows the total required stirrup area. Only at Section 0, where radial tension stirrups are needed, and at Sec. 3, where diagonal tension stirrups are needed, do we find non-zero entries for stirrups.

123. This information is sufficient for the complete design of the conduit wall. A cross section of the designed wall according to these results is shown in Fig. 4-7. Lastly, we should note that the design is somewhat limited by the restriction of the analysis to symmetrically reinforced sections, which is in conflict with the differing inside and outside steel requirements. We do not believe that the final design is greatly affected by this defect, which should be remedied in a final version of the program.



Design Ex. 4.1 Oblong Pipe		
Reinforcement Drawing		Fig. 4-7.
Scale	1:30 1:5	

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PART V: THICK VERSUS THIN: DESIGNS AND COMPARISONS

Introduction

124. In a previous chapter, it was pointed out that consideration of soil-structure interaction in conduit design could be summarized with the question "thick versus thin?". In the following, we will use the analysis capability developed in Part III and the design procedure outlined in Part IV to design a conduit under realistic loading conditions with different wall thicknesses in order to demonstrate how this variable affects the conduit design.

125. It was also pointed out that the conduit shape can affect the structure behavior within the soil, and an additional design of a conduit of improved shape will document this effect. Prior to these designs, we present an analysis to document the validity of our solutions by comparison with published results. Table 5-1 shows the designs carried out in this chapter for the above purposes.

Design Examples

Example 5.1: Analysis Check

126. The purpose of this example is to check the validity of our analysis by comparison with the results presented for Example Problem 3, Page 71, of Ref. 5-1. This is an oblong conduit section under uniformly distributed vertical and lateral loading as shown in Fig. 5-1. No concrete cracking, nor soil-structure interaction, was considered in this linearly-elastic analysis; in the absence of these effects, this can only be considered a partial check.

Example No.	Type of Conduit	Thickness [in]	Applied Load	Soil Stiff. [ksi/in]	Cracking allowed	Comp. Springs
4.1	OBLONG	t = 36	CT,CS,CB	k = 0.	NO	NO
4.2	OBLONG	t = 12	PT,LS,DW	k = .1	YES	YES
4.3	OBLONG	t = 24	PT,LS,DW	k = .1	YES	YES
4.4	OBLONG	t = 36	PT,LS,DW	k = .1	YES	YES
4.5	OVAL	t = 12	PT,LS,DW	k = .1	YES	YES

Table 5-1. Design examples

CT - Const. vertical Top pressure
CS - Const. horizontal Side pressure
CB - Const. vertical Base pressure

PT - Parabolic vertical Top pressure
LS - Linear horizontal Side pressure
DW - Dead Weight of menber

Example No.	Thickness [in]	Critical Section	Mu [k-in/ft]	As [in2/ft]	Stirrups diag. Ten.	Stirrups radial Ten.
4.2	t = 12	0	+206.97	.80	no	no
		5	-215.58	.80	no	no
4.3	t = 24	0	+431.36	.89	no	no
		5	-318.50	.89	no	no
4.4	t = 36	0	+569.84	.98	no	no
		6	-407.13	.98	no	no
		15	+503.93	.98	no	no

Table 5-2. Critical section

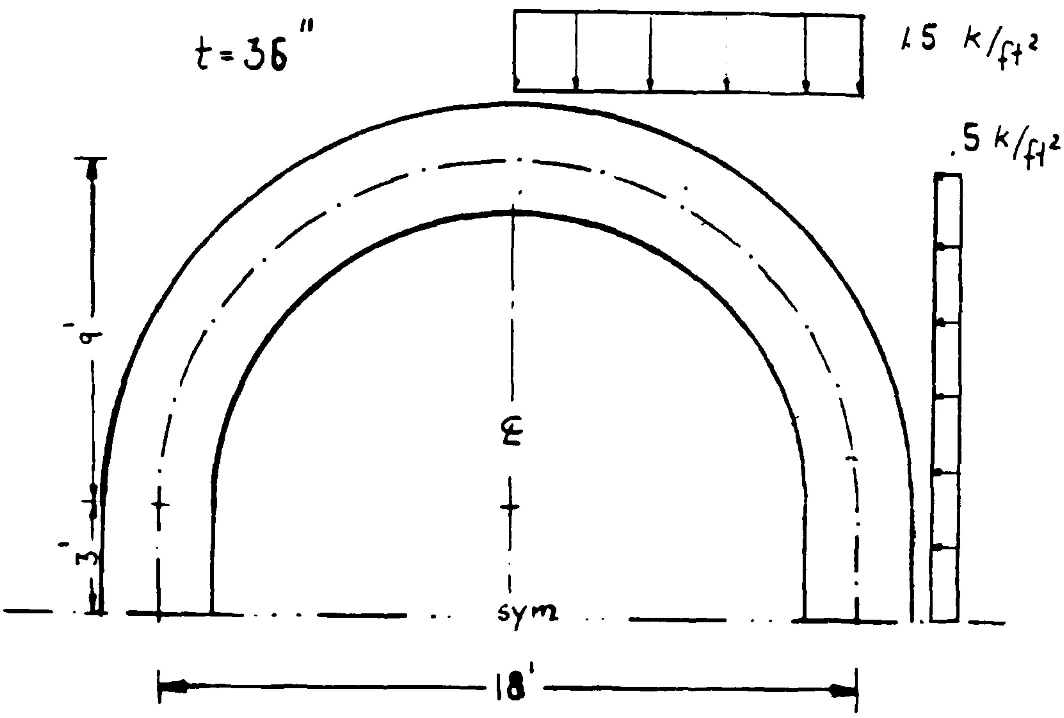

Fig. 5-1. Design example 5.1: thick-walled conduit

127. Fig. 5-2 shows the moment variation according to the analysis of Ref. 5-1, and according to the current analysis. Only a quarter of the structure is plotted due to the double symmetry of this problem. The agreement appears satisfactory.

Example 5.2 to 5.4: Effect of Wall Thickness

128. In these comparison designs, we refer to Example Problem 4, Page 78, of the Harter, Bircher, and Wilson Report (5-1), in which an oblong conduit section, identical to that of our Example 5.1, was subjected to soil pressures shown in Fig. 5-3. Following classical procedures of analysis, working stress design, and neglecting soil-structure interaction, Harter et al. arrived at a wall thickness of 36 inches, thus their design qualifies as a "thick-walled" or "rigid" conduit. In our comparison designs, we will reanalyze and design this culvert for wall thicknesses of 12, 24, and 36 inches, considering soil-structure interaction and following the design procedures of Part IV. A moderate soil stiffness of 0.1 ksi/inch was assumed in all these examples. The resulting designs should provide information for feasibility studies which consider constructibility and economy of these alternates.

129. Rather than present complete analysis results, we will concentrate on the factors necessary for design. Fig. 5-4 shows the moment variation for the three different thicknesses, indicating the radical decrease of moments for the thinner sections. In addition, the moments from Example 4 of Ref. 5-1 are shown for comparison. In the top half of the conduit, they are almost identical to those of our analysis for the 36 inch section, indicating that this rigid conduit hardly feels the surrounding soil. The moments in the lower

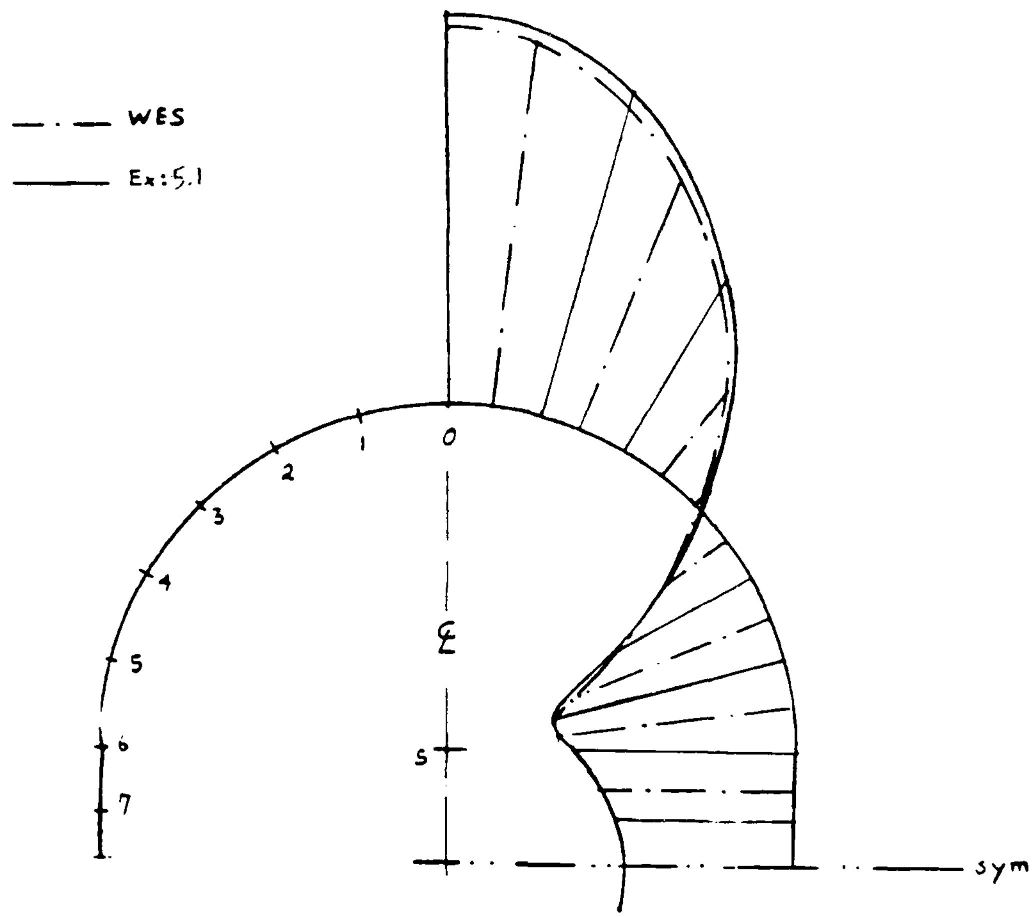

Fig. 5-2. Design example 5.1: Moments according to Ref. 1
and current analysis

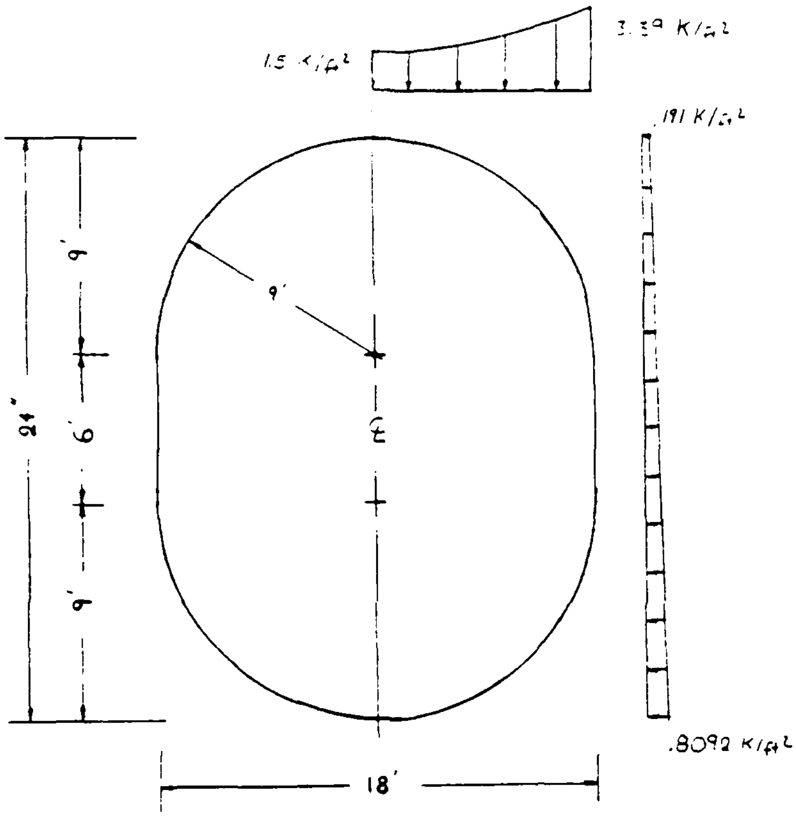

Fig. 5-3. Design examples 5.2 to 5.4

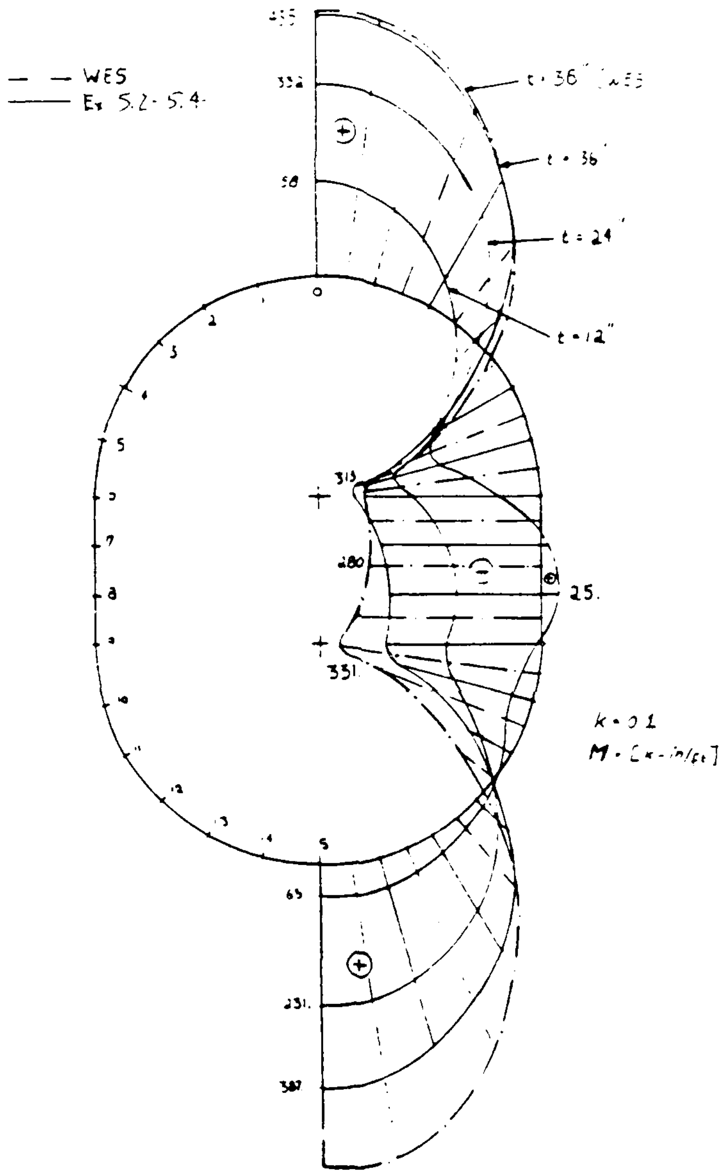

Fig. 5-4. Moments, design examples 5.2 to 5.4

half differ because of different support conditions assumed in Ref. 5-1 and our analysis.

130. Table 5-2 shows the values of critical positive and negative moments for all three wall thicknesses in Column 4, and the required steel area to resist this moment along with thrust in Column 5. In fact, the detailed design results shown in Appendix B indicate that the minimum steel requirements control all of these designs. Columns 6 and 7 indicate that no stirrups are required for any of these designs according to the specifications.

131. The concrete and steel requirements, in square inches per foot, are shown as function of wall thickness in Fig. 5-5. These results clearly indicate the material savings for the thinner-walled structures.

Example 5.5: Effect of Conduit Shape

132. The oblong conduit section of the preceding examples consists of circular ends connected by straight segments. This does not appear an optimal section for arch action, because the straight segments will be unable to resist the soil pressures by membrane action. The discontinuities between straight and circular segments may also be sources of unfavorable moment peaks. To explore these effects, we will in this example consider a smooth elliptical conduit of the same span and width dimensions as the preceding sections, and of wall thickness 12 inches, shown in Fig. 5-6. The results of this analysis will be compared to those of the matching conduit of Example 5.2.

133. The resulting moments for the two sections are shown in Fig. 5 They show indeed that less bending occurs in the elliptical section than in

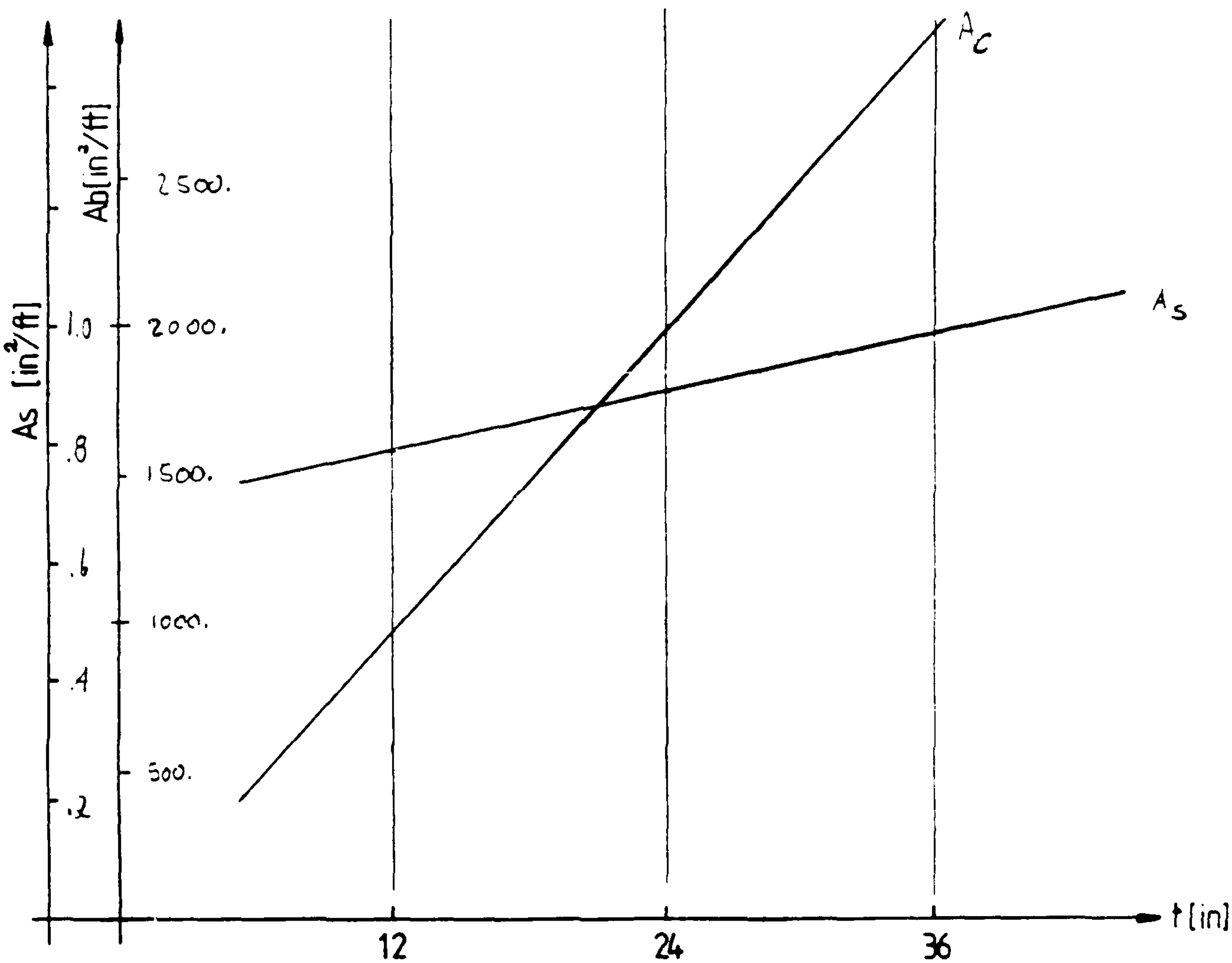

Fig. 5-5. Concrete and steel requirements as function of wall thickness

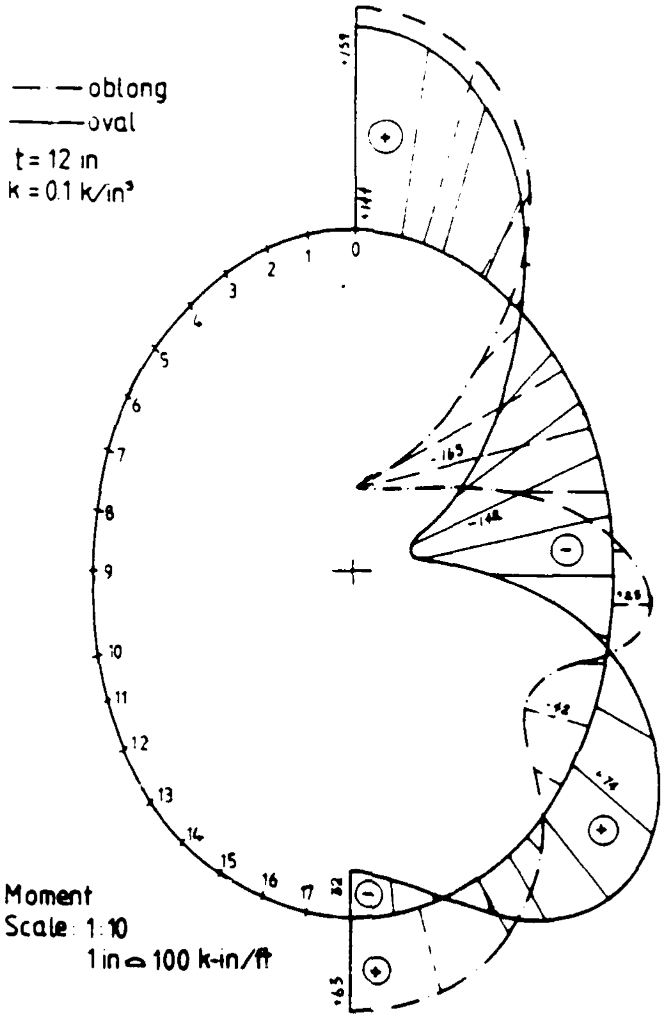

Fig. 5-6. Elliptical conduit section and moments

the oblong section. In the latter, the moment variation in the straight segment is that of a straight beam with end constraints. The moral of this story appears to be that, as in any rational design, attention should be paid to the selection of an optimal shape for the structure.

Reference

5-1. Harter, M.M., Bircher, B.E. and Wilson, A.B., "User's Guide: Computer Program for Design/Review of Curvilinear Conduits/Culverts", Final Report, U.S. Army Corps of Engineers, Feb. 1980.

PART VI: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Summary

134. In this report, we have proposed one way of incorporating recent findings and improvements in structural engineering practice into the design of reinforced concrete culverts. In particular, we have directed attention toward ways of achieving potential savings by considering the surrounding soil as part of the structure, and in this way assessing the help which the soil renders the concrete culvert by carrying part of the load. We have tried to show by analysis and example how reduced concrete wall thickness will lead to culvert pipe of greater efficiency. We have termed this basic issue as "thick versus thin" in culvert design.

135. Another major question which we have tried to address is ways of incorporating the strength method into the design of concrete conduits. After reviewing available literature and codes, we outline a set of specifications based on the 1988 revision of the AASTHO Specifications for the design of concrete culverts. We decided on this course of action in the belief that unification, rather than duplication, of design procedures would be preferable both from the viewpoint of development and from the view of application by design professionals.

136. These specifications leave a good number of questions to be answered, which we have indicated, but they do seem to represent the state of the art at this stage. Considering these two major aspects of conduit design, we have developed a simple analysis program and a design program which we have used for a number of design exercises, both to demonstrate available analysis-design capabilities, and to explore possible economies from using these

rational design approaches. The results of these comparison designs illustrate the potential advantage of thin-walled conduits as compared to the thick-walled sections in current use.

Conclusions

137. The following are the major conclusions from this study:

1. Soil-structure interaction should be included in the design of underground conduits. The soil-structure stiffness ratio is the most important parameter governing this interaction: the larger the value of this parameter, the greater the contribution of the soil to the conduit strength (Part III).

2. It follows from Point 1 that thinner-walled conduits will allow the soil to share the load, thus leading to more effective structures. This is demonstrated in several sample analyses (Parts III and V).

3. If the soil forms part of the structure, greater control of soil properties and placement during construction will be necessary.

4. Analysis of soil-structure interaction is a non-linear problem. Relatively simple models and analysis are possible, but must be checked against both field measurements and refined analysis to establish their validity. This was not done in the present study (Part III).

5. Geotechnical expertise is required for adequate modeling of the soil response. This expertise was not available for this study, thus the assumptions on soil behavior in the current analysis need verification.

6. The design rules of Chapter 17 of the 1988 revision of the AASTHO Specifications seem to represent the current state of the art, and were followed in the design studies of this report. However, they require

verification and improvement in a number of points (Part IV).

7. Rational analysis is shown to be a valuable tool in verifying and extending the simple empirical conduit design rules suggested by CALTRANS. (Part III, Example 3).

8. Comparison designs demonstrate the material economies possible with consideration of soil-structure interaction (Part V).

Recommendations for Further Work

138. This study had as its aim the updating of U.S. Corps of Engineers procedures for conduit design, and we believe that it lays the groundwork to this purpose with suggestions for basic approaches, implementation, and design applications. However, the scope of this assignment forced us to restrict ourselves to the essentials. Further work necessary before these approaches can be implemented includes the following:

1. Review of the assumptions regarding soil behavior by geotechnical experts.

2. Critical comparisons of analysis results from simple and complex models in order to arrive at procedures which strike a happy medium between reality and simplicity. In particular, results from simple framed-structure models should be compared to those from finite-element models such as SPIDA, or similar programs.

3. A user-friendly analysis program for design office use will be required.

4. A review of the suggested safety and serviceability criteria in the Specifications is necessary. Performance records of as-built conduits would be helpful to this end.

5. The sections of the specifications dealing with flexure, cracking, and diagonal tension need critical review, as detailed in Part IV. The entire specifications should be reviewed by a team of experts and professionals to insure their usefulness for design practice.

6. Construction methods and field control provisions necessary to insure the assumed soil-structure interaction should be established by geotechnical and construction experts.

7. The method of accounting for soil-structure interaction suggested by CALTRANS should be subjected to thorough analysis and field testing.

APPENDIX A - COMPUTER PROGRAM FOR CONDUIT DESIGN

The listing of Program PIPEDESIGN which is supplied in this appendix is based on Eqs. 4.1 to 4.10 of the Specifications of Part IV. Its documentation is considered sufficient for use, but lays no claim to being a user-friendly manual.

This program was used for the design of Example Designs 4.1, as well as 5.1 to 5.5. For the former, output sheets are contained in Part IV, for the latter, in Appendix B.

```
C***************************************************************************

      PROGRAM PIPEDESIGN

C***************************************************************************

C     PURPOSE:  TO DESIGN BURIED REINFORCED CONCRETE PIPES ACCORDING
C               TO THE AASHTO SPECIFICATIONS FOR HIGHWAY BRIDGES,
C               SEE CHAPTER 17.
C
C     INPUT PARAMETER:  SEE SUBROUTINE "INLIST"
C                       N - ARRAY DIMESNION VARIABLE
C
C     OUTPUT PARAMETER: SEE SUBROUTINE "OUTLIST"
C
C     SUBROUTINES CALLED: INLIST
C                         FEXREIN
C                         CRACKCON
C                         STIRRUP
C                         OUTLIST
C                         NUPAGE
C
C     PROGRAM LANGUAGE:   FORTRAN 77
C
C     AUTHOR: RAUSCHER THOMAS, BOULDER JULY 1988
C             VERSION 1.

C***************************************************************************

      IMPLICIT REAL*8 (A-H,O-Z)

      DIMENSION CI(3),BI(3,20),SN(20),SM(20),SV(20),
     &          UV(20),UN(20),UM(20),NOSEC(20),ASF(20)

      DIMENSION ASMINI(20),ASMINO(20),NCOF(20),
     &          ASMAXC(20),ASMAXR(20),ASRQ(20)

      DIMENSION AVS(20),AVR(20),SMAX(20),
     &          VB(20),VC(20)

      CHARACTER*100 INFO

      PARAMETER (IN=2,IOU=3,N=20)

C-----ASIGN REINFORCEMENT COEFFICIENT
C     smooth wire
      CI(1) = 1.0
C     welded deformed wire
      CI(2) = 1.9
C     welded smooth wire
      CI(3) = 1.5

C     READ INPUTDATA
C--------------------------------------------------------------------------

      CALL INLIST (IN,B,H,S,R1,THI,STRA,FC,FY,DC,TB,KSEC,NOSEC,
     &             SM,SN,SV,CRW,PHIF,PHIV,INFO,N,RLO)

C--------------------------------------------------------------------------

C-----DISTANCE FROM COMP. FACE TO CENTROID OF TENSION BARS
      D = H-DC
      R = (S/2.) - (H/2.)
      RS = R + DC

C-----ULTIMATE INTERNAL FORCES
      DO 100 I=1,KSEC
         UV(I) =  SV(I)*RLO
         UN(I) = -SN(I)*RLO
         UM(I) =  SM(I)*RLO
         SN(I) = -SN(I)
 100  CONTINUE

C-----CALCULATE MAX. ULT. SHEAR STRENGHT FOR DIA.TENSION

      VCMAX = 2.*PHIV * D * B * DSQRT(FC)


C     CALCULATE REINFOREMENT FOR FLEXIBLE STRENGTH
C--------------------------------------------------------------------------

      CALL FLEXREIN (N,KSEC,D,H,S,B,RS,FC,FY,PHIF,SM,UN,UM,
     &               ASF,ASMINI,ASMINO,ASMAXC,ASMAXR,IFAIL)

C--------------------------------------------------------------------------

      IF (IFAIL .EQ. 1) THEN
         PRINT*,'FLEXREIN IFAIL =',IFAIL
         GOTO 99
      END IF

      DO 200 K=1,KSEC

C-----CHECKING OF REQUIRED REINFORCMENT
      ASRQ(K) = ASF(K)

      IF(ASMINI(K) .GT. ASMINO(K)) THEN
         ASMIN = ASMINI(K)
      ELSE
         ASMIN = ASMINO(K)
      END IF

      IF (ASMIN .GT. ASRQ(K)) ASRQ(K) = ASMIN
C

C-----CHECKING COMPRESSION CONCRETE FAILURE

         NCOF(K) = -1
      IF ( ASMAXC(K) .LT. ASRQ(K)) THEN
         NCOF(K) = NOSEC(K)
      END IF


C     CHECK CRACK WIDTH
C--------------------------------------------------------------------------

      CALL CRACKCON (N,K,B,H,D,CRW,TB,FC,FY,SM,SN,ASRQ,
     &               CI,BI,NOSEC)

C--------------------------------------------------------------------------

C     SHEAR REINFORCEMENT
C--------------------------------------------------------------------------

      CALL STIRRUP (N,K,B,PHIV,D,R,RS,FC,FY,SM,UM,UN,UV,
     &              ASRQ,ASMAXR,AVS,AVR,VB,VC,VCMAX,SMAX)

C--------------------------------------------------------------------------


 200  CONTINUE


C     CREATE OUTPUT FILE
C--------------------------------------------------------------------------

      CALL OUTLIST (IOU,N,KSEC,S,B,H,STRA,FC,FY,PHIF,PHIV,TB,I ,
     &              CRW,RLO,SM,SN,SV,UM,UN,UV,ASF,ASRQ,ASMINI,
     &              ASMINO,ASMAXR,ASMAXC,VC,AVR,SMAX,AVS,VB,
     &              BI,NOSEC,NCOF,INFO)

C--------------------------------------------------------------------------

C---- END OF PROGRAM PIPEDESIGN ---

 99   STOP
      END
```

```
C***********************************************************************
      SUBROUTINE INLIST (IN,B,H,S,R1,THI,STRA,FC,FY,DC,TB,KSEC,NOSEC,
     *                   SM,SN,SV,CRW,PHIF,PHIV,INFO,N,RLO)

C***********************************************************************
C
C     PURPOSE:  READ DATA FROM FILE  >DESIGN.IN< TO RUN PIPEDESIGN
C               PROGRAM
C
C     INPUT PARAMETER:  IN
C
C     OUTPUT PARAMETER: B,H,S,R1,THI,STRA,FC,FY,DC,TB,KSEC,NOSEC,
C                       SM,SN,SV,SN,SV,CRW,PHIF,PHIV,RLO
C
C     SUBROUTINES CALLED: NONE
C
C     CALLED BY:            PIPEDESIGN
C
C     PROGRAM LANGUAGE:     FORTRAN 77
C
C     AUTHOR: RAUSCHER THOMAS, BOULDER JULY 1988
C             VERSION 1.
C
C***********************************************************************

      IMPLICIT REAL*8 (A-H,O-Z)

      DIMENSION SM(20),SN(20),SV(20),NOSEC(20)

      CHARACTER*100 INFO

C-----OPEN INPUT UNIT AND READ DATA
      OPEN (UNIT=IN,FILE='de: in')

C-----USER DEFINED TEXTLINE IN APOSTROPHIES(70 LETTERS)

      READ(IN,*) INFO

C-----CROSSSECTION DATA
C     B - LENGHT OF MEMBER [in]
C     H - OVERALL THICKNESS OF MEMBER [in]
C     S - SPAN OF PIPE [in]

      READ(IN,*) B,H,S

C-----MATERIAL PROPERTIES
C     FC   - COMPRESSION STRENGTH OF CONCRETE [ksi]
C     FY   - YIELD STRENGTH OF REINFORCEMENT (NONPRESTRESSED) [ksi]
C     STRA - STEEL RATIO OF THE  PREVIOUS RUN [%]
C     DC   - DISTANCE FROM THE TENSION FIBER TO THE CENTER OF THE BARS [in]
C     TB   - CLEAR COVER OF THE BARS [in]

      READ(IN,*) FC,FY,STRA,DC,TB
           FC = FC*1000.
           FY = FY*1000.


C-----INTERAL FORCES AT THE CRITICAL SECTION
C     KSEC - NO  OF SECTIONS TO BE CONSIDERED
C     RLO  - LOAD FACTOR FOR ULTIMATE INTERNAL FORCES

      READ(IN,*)KSEC,RLO

      DO 100 K=1,KSEC

C     NO - SECTION NUMBER

      READ(IN,*) NOSEC(K)

C     SM - BENDING MOMENT UNDER SERVICE LOAD [k-in/ft]
C     SN - AXIAL FORCE UNDER SERVICE LOAD [Kips/ft],comp.(-)
C     SV - SHEAR FORCE      "       "       "      "

      READ(IN,*) SM(K),SN(K),S'(K)
           SM(K) = SM(K)*1000.
           SN(K) = SN(K)*1000.
           SV(K) = SV(K)*1000.

  100 CONTINUE

C-----USER DEFINED VALUES
C     CRW  - MAX. ALLOWABLE CRACK WIDTH [in]
C     PHIF - STRENGHT REDUCTION FACTOR FOR BENDING
C     PHIV -     "          "        "    FOR SHEAR

      READ(IN,*) CRW,PHIF,PHIV

C---- END OF INLIST FILE ----
      RETURN
      END
```

```
C***********************************************************************

      SUBROUTINE FLEXREIN (N,KSEC,D,H,S,B,RS,FC,FY,PHIF,SM,[illegible]M,
     &                     ASF,ASMINI,ASMINO,ASMAXC,ASMAXR,IFAIL)

C***********************************************************************
C
C     PURPOSE:  TO OBTAIN THE REQUIRED ,MIN. AND MAX. REINFORCEMENT
C
C     INPUT PARAMETER:  D,H,S,B,R1,FC,FY,PHIF,UN,UM,SM,N,KSEC
C
C     OUTPUT PARAMETER: ASF - TENSION REINFORCEMENT AREA FOR FLEX.STRENGTH
C                       ASMINI - MIN. REINFORCEMENT ON THE INSIDE
C                       ASMINO - MIN. REINFORCEMENT ON THE OUTSIDE
C                       ASMAXC - MAX. REIN. LIMITED BY CONCRETE COMP.
C                       ASMAXR - MAX. REIN. LIMITED BY RADIAL TENSION
C                       PHIF   - CAPACITY REDUCTION FACTOR FOR SM & SN
C                       IFAIL  - ERROR INDICATOR (1 = ERROR)
C     SUBROUTINES CALLED: NONE
C
C     CALLED BY:          PIPEDESIGN
C
C     PROGRAM LANGUAGE:   FORTRAN 77
C
C     AUTHOR: RAUSCHER THOMAS, BOULDER JULY 1988
C             VERSION 1.

C***********************************************************************

      IMPLICIT REAL*8 (A-H,O-Z)

      DIMENSION SM(N),UM(N),UN(N),ASF(N)

      DIMENSION ASMINI(N),ASMINO(N),
     &          ASMAXC(N),ASMAXR(N)

C-----REINFORCEMENT FOR FELXURAL STRENGTH
      GMAX = .85 * B * FC
      A    = PHIF * D
      BO   = 2.0 * PHIF * D - H
      AG   = GMAX * A

C-----LOOP OVER EVERY SECTION-----------------------------
      DO 100 K=1,KSEC

      ROOT = DSQRT(GMAX* (AG*A - UN(K)*BO - 2.*DABS(UM(K))))
      ASF(K)  = ( AG - UN(K) - ROOT) / FY
      IF(ASF(K) .LT. 0) ASF(K) = 0.
C
C-----MINIMUM REINFORCMENT
      ASMIN  = (S + H)**2 / 65000.
      IF (SM(K) .LT. 0) THEN
          ASMINO(K) = ASMIN
          ASMINI(K) = .75 * ASMIN
      ELSE
          ASMINI(K) = ASMIN
          ASMINO(K) = .75 * ASMIN
      END IF
C
C-----CHECK MIN. REQUIREMNET 0.07 in2/ft
      IF(ASMINI(K) .LT. 0.07) ASMINI(K) = 0.07
      IF(ASMINO(K) .LT. 0.07) ASMINO(K) = 0.07

  100 CONTINUE

C
C-----MAX. FLEX. REINFORCEMENT LIMITED BY CONRCETE COMPRESSION
      GMIN  = .65*B*FC
      C     = (FC - 4000.) / 1000.

      GST = B * FC *(.85 - .05*C)
      IF( GST .LT. GMIN) GST = GMIN
      IF( GST .GT. GMAX) GST = GMAX

      DO 200 K=1,KSEC

      ASMAXC(K) = (((55000.*GST*A / (87000.+FY)) - .75*UN(K))) / FY
C
C-----LIMITED BY RADIAL TENSION

      ASMAXR(K) = 16.* RS * DSQRT(FC) / FY

  200 CONTINUE

C---- END OF FLEXREIN ----
      RETURN
      END
```

```
C************************************************************************

      SUBROUTINE CRACKCON (N,K,B,H,D,CRW,TB,FC,FY,SM,SN,ASRQ,CI,BI,
     *                     NOSEC)

C************************************************************************
C
C     PURPOSE:  TO CONTROL THE CRACK WIDTH UNDER SERVICE LOAD DESIGN
C
C     INPUT PARAMETER:  B,H,D,CRW,SM,SN,TB,ASRQ,CI,N, NOSCE,KSEC,
C                       FC,FY,SM,SM,
C
C     OUTPUT PARAMETER: BI(1) - PLAIN BAR SPACING/NO.OF BARS
C                       BI(2) - WELDED DEFORMED SPCACING/NO.OF BARS
C                       BI(3) - ASF FOR WELDED SMOOTH WIRES
C
C     SUBROUTINES CALLED: NONE
C
C     CALLED BY:           PIPEDESIGN
C
C     PROGRAM LANGUAGE:    FORTRAN 77
C
C     AUTHOR: RAUSCHER THOMAS, BOULDER JULY 1988
C             VERSION 1.

C************************************************************************

      IMPLICIT REAL*8 (A-H,O-Z)

      DIMENSION CI(3),BI(3,N),SN(N),SM(N)

      DIMENSION ASRQ(N),NOSEC(N)

C-----ECCENTRICITY
      EC    = DABS(SM(K)/SN(K)) + D - (H/2.)
      ED    = EC/D
C-----CHECK FOR CRACKING
      IF ( ED .LT. 1.15 ) THEN
         PRINT*,'NO CRACKING AT POINT ',NOSEC(K)
         RETURN
      END IF

      AJ    = .74 + .1*EC/D
      IF (AJ .GT. .9) AJ = .9
      AI    = 1./(1.-(AJ*D/EC))
      E     = DABS(SM(K)) + SN(K)*(D-H/2.)

      FCR = CRW/0.01

C-----CHECK THE APPLICATION RANGE
      IF(FCR .LT. 0.7 .OR. FCR .GT. 1.7) THEN
        PRINT*,'CRACK CONTROL CAN NOT BE USED'
        PRINT*,'ALLOWABLE CRACK WIDTH OUT OF RANGE'
        RETURN
      END IF

      TBT = TB*TB /2.

       DO 100 I=1,3
        C1 = CI(I)
        BRAN = E/(AJ*AI) - (C1 * B * H*H * DSQRT(FC))
        IF ( BRAN .LE. 0) THEN
           PRINT*,' CRACK CONTROL NOT DECISIVE AT ',NOSEC(K)
           BI(1,k) = 0
           BI(2,k) = 0.
           BI(3,k) = 0.
           GOTO 99
        END IF

          RATE = FCR * ASRQ(K) * D * 30000./BRAN
          B1   = RATE**3
          BI(I,K) =  B1 / TBT
          IF( C1 .EQ. 1.5) THEN
          BI(I,K) =  BRAN / (30000.*D*FCR)
          END IF

  100   CONTINUE

  99    RETURN
       END
```

```
C*************************************************************************

      SUBROUTINE STIRRUP  (N,K,B,PHIV,D,R,RS,FC,FY,SM,UM,UN,UV,
     &                     ASRQ,ASMAXR,AVS,AVR,VB,VC,VCMAX,SMAX)

C*************************************************************************
C
C     PURPOSE:  TO CHECK FOR ADEQUACY SHEAR STRENGTH
C
C     INPUT PARAMETER:  B,PHIV,ASRQ,D,R,RS,UM,UN,UV,
C
C     OUTPUT PARAMETER: AVR - REQUIRED AREA OF STIRRUPS REINFORCEMENT
C                             FOR RADIAL TENSION
C                       AVS - REQUIRED AREA OF STIRRUPS FOR SHEAR REIN.
C                       SMAX- MAX. SPACING FOR STIRRUPS
C                       VB  - MAX. SHEAR STRENGHT WITHOUT STIRRUPS
C                       VC  - ULTIMATE SHEAR FORCE WHICH PRODUCTS DIAGONAL
C                             TENSION FAILURE WITHOUT STIRRUPS
C
C     SUBROUTINES CALLED: NONE
C
C     CALLED BY:          PIPEDESIGN
C
C     PROGRAM LANGUAGE:   FORTRAN 77
C
C     AUTHOR: RAUSCHER THOMAS, BOULDER JULY 1988
C             VERSION 1.

C*************************************************************************

      IMPLICIT REAL*8 (A-H,O-Z)

      DIMENSION SM(N),UM(N),UN(N),UV(N),ASRQ(N),ASMAXR(N)

      DIMENSION AVS(N),AVR(N),VB(N),VC(N),
     &          SMAX(N)

      FD = .8 + 1.6/(PHIV*D)

      IF (FD .LT. 1.25) FD = 1.25

      TOT = (PHIV*D)/(2.*R)
C
C-----CHECK FOR TENSION SIDE
      IF (SM(K) .LT. 0) THEN
         FFC = 1. - TOT
      ELSE
         FFC = 1. + TOT
      END IF

      IF(UV(K) .NE. 0) THEN
        FN = 1.0 - .12*( UN(K)/DABS(UV(K)) )
      ELSE
        FN = 0.
      END IF

      IF (FN .LT. .75) FN = .75

C-----CHECK FOR STEEL RATIO ROH
      ROH  = ASRQ(K) / (PHIV*B*D)
      FACT = FD / (FFC*FN)
      ROMIN = 200. / FY
      ROMAX = 0.02
C
      IF(ROH .LT. ROMIN) ROH = ROMIN
      IF(ROH .GT. ROMAX) ROH = ROMAX

C-----BASIC SECTION SHEAR STRENGTH Vb

      VB(K) = B*PHIV*D*DSQRT(FC)*(1.1+63.*ROH)*FACT

      IF(ASRQ(K).LT. ASMAXR(K)) GOTO 33

C-----RADIAL STIRRUPS / SPACING

      BOT     = (DABS(UM(K)) - .45*UN(K)*PHIV*D)
      SMAX(K) = .75*PHIV*D
      AVR(K)  = (1.1*BOT) / (FY*RS*PHIV*D)

 33   IF (AVR(K) .LE. 0) AVR(K) = 0.

C-----SHEAR STIRRUPS -RADIAL + DIAGONAL TENSION

      IF(UV(K) .EQ. 0) THEN
         VC(K)  = VCMAX
         AVS(K) = AVR(K)
         RETURN
      END IF

      DNOM     = DABS(UV(K))*PHIV*D

      IF(DNOM .EQ. 0) THEN
         TOT   =  DABS(UV(K))*FFC
         GOTO 77
      ELSE
         DNO   = (DABS(UM(K)) / DNOM) + 1.
         VC(K) = 4.*VB(K) / DNO
      END IF

      IF(VC(K) .GT. VCMAX) VC(K) = VCMAX

         TOT   =  (DABS(UV(K))*FFC - PHIV*VC(K))


 77   IF(VB(K) .GT. DABS(UV(k))) GOTO 44

      AVS(K)  = (1.1 / (FY*PHIV*D)) * TOT + AVR(K)

 44   IF (AVS(K) .LE. 0)  AVS(K)  = AVR(K)


      RETURN
      END
```

```
C**********************************************************************

      SUBROUTINE OUTLIST(IOU,N,KSEC,S,B,H,STRA,FC,FY,PHIF,PHIV,TB,DC,
     &                   CRW,RLO,SM,SN,SV,UM,UN,UV,ASF,ASRQ,ASMINI,
     &                   ASMINO,ASMAXR,ASMAXC,VC,AVR,SMAX,AVS,VB,
     &                   BI,NOSEC,NCOF,INFO)

C**********************************************************************

C     PURPOSE:  TO CREATE AN OUTPUT FILE CALLED 'des.out' WHICH
C               CONTAINS INPUT- AND OUTPUT VALUES
C
C     INPUT PARAMETER:  SEE SUBROUTINE "INLIST"
C
C     OUTPUT PARAMETER: SEE WRITE COMMANDS
C
C     SUBROUTINES CALLED: NUPAGE
C
C     PROGRAM LANGUAGE:   FORTRAN 77
C
C     AUTHOR: RAUSCHER THOMAS, BOULDER JULY 1988
C             VERSION 1.

C**********************************************************************

      IMPLICIT REAL*8 (A-H,O-Z)

      DIMENSION CI(3),BI(3,N),SM(N),SN(N),SV(N),
     &          UV(N),UN(N),UM(N),NOSEC(N),ASF(N)

      DIMENSION ASMINI(N),ASMINO(N),NCOF(N),
     &          ASMAXC(N),ASMAXR(N),ASRQ(N)

      DIMENSION AVS(N),AVR(N),SMAX(N),
     &          VB(N),VC(N)

      CHARACTER*100 INFO

      OPEN (UNIT=IOU,FILE='des.out')

      PRINT*,'WRITE OUTPUT DATA INTO FILE: des.out'

      WRITE(IOU,1)
 1    FORMAT(/,15X,'*******************************************',/
     &       15X,'*  P R O G R A M   P I P E - D E S I G N  *',/
     &       15X,'*******************************************',//)

      WRITE(IOU,7) INFO
 7    FORMAT(5X,100A)

      AB=B*H
      WRITE(IOU,2) S,B,H,AB
 2    FORMAT(//,5X,'CROSS SECTION :',/,5x,'---------------',//,
     &       5X,'Pipe Width       s =',F10.2,' [in]',/,
     &       5x,'Segment Length   b =',F10.2,' [in]',/,
     &       5X,'Thickness        t =',f10.2,' [in]',/,
     &       5x,'Cross Section   Ab =',f10.2,' [in2]',/)

      WRITE(IOU,5) STRA,TB,DC,CRW
 5    FORMAT(5X,'Steel Ratio        roh  =',f10.2,' [%]',/,
     &       5x,'Clear Cover         tb =',f10.2,' [in]',/,
     &       5x,'CL Steel Cover      dc =',f10.2,' [in]',/,
     &       5x,'all. Crack Width   crw =  ',f8.3,'  [in]',//)

       WRITE(IOU,88) PHIF,PHIV,RLO
 88    FORMAT(5X,'SAFETY FACTORS :',/,5x,'--------------',//,
     &       5x,'Reduction Factor phif =',f10.2,' [/]',/,
     &       5x,'                 phiv =',f10.2,' [/]',/,
     &       5X,'Load Factor       rlo =',f10.2,' [/]',//)

      WRITE(IOU,3) FC/1000.,FY/1000.
 3    FORMAT(5X,'MATERIAL PROPERTIES  :',/
     &      ,5x,'---------------------',//,
     &       5x,'Compression Strength  fc =',f10.2,' [ksi]',/,
     &       5x,'Yield Strength Steel  fy =',f10.2,' [ksi]',//)

      IF(KSEC .GT. 6) CALL NUPAGE(IOU)

      WRITE(IOU,6)
 6    FORMAT(//15X,' > D E S I G N  -  R E S U L T S <',/
     &          15X,' ******************************** ',//)

      ITAB = 1

      WRITE(IOU,44) ITAB
 44   FORMAT(//,5X,'Table ',I1,' : Section Features',/)

      ITAB = ITAB + 1

      WRITE(IOU,4)
 4    FORMAT(/,9X,'1',10X,'2',13X,'3',14X,'4',14X,'5',11X,'6',/
     &          ,5X,' Section |       Mu      |       Nu        |',
     &'      Vu      |  req. Steel  |  Ratio  |',/,
     &          5x,'   No.   | [k-in/ft]    |    [kips/ft]    |',
     &'   [kips/ft]  |  As [in2/ft] | roh [%] |',/,
     &5x,'---------|-------------|---------------|--------------|',
     &'--------------|---------|',/)

      DO 100 K=1,KSEC

      WRITE(IOU,40) NOSEC(K),UM(K)/1000.,-UN(K)/1000.,UV(K)/1000.,
     &              ASRQ(K),(ASRQ(K)/AB)*100.
 40   FORMAT(7X,I2,5X,'|',SP,F10.3,2X,'|',1X,SP,F10.3,3X,'|',
     &              2X,SP,F10.3,2X,'|',3X,F7.3,4X,'|  ',f6.3,' | ',/)

 100  CONTINUE

C----------------------
      CALL NUPAGE (IOU)
C----------------------

      WRITE(IOU,66) ITAB
 66   FORMAT(//,5X,'Table ',I1,
     &' : Reinforcement for Flexural Strength',/)

      ITAB = ITAB +1

      WRITE(IOU,8)
 8    FORMAT(/,9X,'1',10X,'2',9X,'3',11X,'4',12X,'5',/
     &          ,5x,' Section |    As     |   Asmin    |   Asmin   |',
     &'   Asmaxc   |',/
     &          ,5x,'   No.   | [in2/ft] |   Inside   |  Outside  |',
     &' Compres.   |',/
     &          ,5x,'---------|----------|------------|-----------|',
     &'------------|',/)


      DO 111 K =1,KSEC

      WRITE(IOU,80) NOSEC(K),ASF(K),ASMINI(K),ASMINO(K)
     &              ,ASMAXC(K)
 80   FORMAT(7X,I2,5X,'| ',F8.3,1X,'| ',F8.3,2X,'| ',F8.3,2X,'  ',
     &              F8.3,3X,'| ',/)

 111  CONTINUE

C-----CHECK CONCRETE COMPRESSION FAILURE
```

```
      DO 54 K=1,KSEC
         IF (NCOF(K) .NE. -1) THEN
         M = 1
         WRITE(IOU,53) NCOF(K)
 53      FORMAT(/,5X,'CAUTION :',
     &   ' CONCRETE COMPRESSION FAILURE at SECTION ',I2,/)
         END IF
 54   CONTINUE

      IF (M.EQ. 1)THEN
        WRITE(IOU,56)
 56     FORMAT(/,15X,'!! BIGGER SECTION THICKNESS REQUIRED !!',)
        GOTO 99
      END IF

      WRITE(IOU,55)  ITAB
 55   FORMAT(////,5X,'Table ',I1,' : Crack Control Design',/)

      ITAB = ITAB +1

      WRITE(IOU,9)
 9    FORMAT(/,9X,'1',11X,'2',12X,'3',15X,'4',/,
     &       5X,' Section |            Type of Reinforcement',
     &'               |',/,
     &       5x,'   No.   |    max. spacing  [in]    |   ',
     &'As [in2/ft]]     |',/,
     &       5x,'         |     Type 1    |     Type 3   |   ',
     &'    Type 2      |',/,
     &       5x,'---------|--------------|-------------|',
     &'-------------------|',/)

      DO 112 K=1,KSEC

      WRITE(IOU,90) NOSEC(K),BI(1,K),BI(2,K),BI(3,K)
 90   FORMAT(7X,I2,5X,'|',1X,F8 2,4X,'|',1X,F8.2,3X,'|',
     &       3X,F8.3,8X,'|',/)

 112  CONTINUE

      WRITE(IOU,95)
 95   FORMAT(/,5X,'Type  1 :  Smooth Wire or Plain Bars',/
     &          ,5x,'Type  2 :  Welded Smooth Wire Fabric',/
     &          ,5x,'Type  3 :  Welded Deformed Wire Fabric',//)

      IF(KSEC .GE. 6) CALL NUPAGE(IOU)

      WRITE(IOU,201) ITAB
 201  FORMAT(//,5X,'Table ',I1,
     &' : Radial and Diagonal Tension Reinforcement ',/)


      WRITE(IOU,202)
 202  FORMAT(/,9X,'1',10X,'2',10X,'3',10X,'4',12X,'5',14X,'6',/,
     &         5x,' Section |  Asmaxr  |     Vb     |     Vc      |',
     &'   Avr/s     |      Avs/s    |',/
     &         ,5x,'   No.   | [in2/ft] | [kips/ft] | [kips/ft] |',
     &' [in/ft]     |    [in/ft]    |',/
     &         ,5x,'---------|----------|-----------|-----------|',
     &'------------|--------------|',/)


      DO 204 K =1,KSEC

      WRITE(IOU,203) NOSEC(K),ASMAXR(K),VB(K)/1000.,VC(K)/1000.,
     &                   AVR(K),AVS(K)

 203  FORMAT(7X,I2,5X,'| ',F8.3,1X,'| ',F8.3,2X,'| ',F8.3,2X,'| ',
     &                   F8.3,3X,'|',2x,F8.3,4x,'| ',/)

 204  CONTINUE

 99   RETURN

      END


c###################################################################

      subroutine nupage(iout)

c###################################################################
c     purpose:
c     this subroutine causes that the following results are writter
c     on new page of output.
c     calling sequence: call nupage
c     arguments:         iout- output uint
c     subroutines call: none
c     called from:       results1
c-------------------------------------------------------------------
      character*1 FORMFD
c
      FORMFD=char(12)
      write(iout,*) FORMFD
c
      return
      end
```

APPENDIX B - OUTPUT LISTINGS FOR EXAMPLES 5.2 - 5.5

In the following, output for the initial run (with steel ratio = 0.004) and one iteration is supplied for each of Design Examples 5.2 to 5.5.

The reader should refer back to Part IV for the basic specifications, to Part V for discussion of these design examples, and to Appendix A for the design program listing.

*** P R O G R A M   P I P E - D E S I G N ***

Design Example 5.2 Oblong Pipe, Soil-Structure-Interaction, k=.1 [k/in3]

CROSS SECTION :

```
Pipe Width       s  =   216.00 [in]
Segment Length   b  =    12.00 [in]
Thickness        t  =    12.00 [in]
Cross Section    Ab =   144.00 [in2]

Steel Ratio      roh  =    0.40 [%]
Clear Cover      tb   =    1.00 [in]
CL Steel Cover   dc   =    1.50 [in]
all  Crack Width crw  =   0.010  [in]
```

SAFETY FACTORS :

```
Reduction Factor phif =    0.90 [/]
                 phiv =    0.85 [/]
Load Factor      rlo  =    1.30 [/]
```

MATERIAL PROPERTIES :

```
Compression Strength  fc =    4.00 [ksi]
Yield Strength Steel  fy =   60.00 [ksi]
```

> D E S I G N - R E S U L T S <

Table 1 : Section Features

1 Section No.	2 Mu [k-in/ft]	3 Nu [kips/ft]	4 Vu [kips/ft]	5 req. Steel As [in2/ft]	6 Ratio roh [%]
0	+199.628	-11.115	+0.000	+0.800	+0.555
5	-215.826	-29.250	-1.186	+0.800	+0.555

Table 2 : Reinforcement for Flexural Strength

1 Section No.	2 As [in2/ft]	3 Asmin Inside	4 Asmin Outside	5 Asmaxc Compres.
0	0.249	0.800	0.600	2.265
5	0.098	0.600	0.800	2.039

Table 3 : Crack Control Design

1 Section No.	2 Type of Reinforcement max. spacing [in] Type 1	3 Type 3	4 As [in2/ft]] Type 2
0	0.00	0.00	0.000
5	0.00	0.00	0.000

Type 1 : Smooth Wire or Plain Bars
Type 2 : Welded Smooth Wire Fabric
Type 3 : Welded Deformed Wire Fabric

Table 4 : Radial and Diagonal Tension Reinforcement

1 Section No.	2 Asmaxr [in2/ft]	3 Vb [kips/ft]	4 Vc [kips/ft]	5 Avr/s [in/ft]	6 Avs/s [in/ft]
0	1.746	16.986	13.547	0.000	0.000
5	1.746	18.540	3.466	0.000	0.000

****

* P R O G R A M   P I P E - D E S I G N *

****

Design Example 5.2 Oblong Pipe, Soil-Structure-Interaction, k=.1 (k/in3)

CROSS SECTION :

```
Pipe Width        s  =    216.00 (in)
Segment Length    b  =     12.00 (in)
Thickness         t  =     12.00 (in)
Cross Section     Ab =    144.00 (in2)

Steel Ratio       roh =     0.56 (%)
Clear Cover        tb =     1.00 (in)
CL Steel Cover     dc =     1.50 (in)
all. Crack Width  crw =    0.010 (in)
```

SAFETY FACTORS :

```
Reduction Factor phif =     0.90 (/)
                 phiv =     0.85 (/)
Load Factor       rlo =     1.30 (/)
```

MATERIAL PROPERTIES :

```
Compression Strength  fc =     4.00 (ksi)
Yield Strength Steel  fy =    60.00 (ksi)
```

>D E S I G N - R E S U L T S<

Table 1 : Section Features

1 Section No.	2 Mu (k-in/ft)	3 Nu (kips/ft)	4 Vu (kips/ft)	5 req. Steel As (in2/ft)	6 Ratio roh (%)
0	+206.973	-11.024	+0.000	+0.800	+0.555
5	-215.582	-29.250	-1.100	+0.800	+0.555

Table 2 : Reinforcement for Flexural Strength

1 Section No.	2 As (in2/ft)	3 Asmin Inside	4 Asmin Outside	5 Asmaxc Compres.
0	0.264	0.800	0.600	2.266
5	0.097	0.600	0.800	2.039

Table 3 : Crack Control Design

1 Section No.	2 Type of Reinforcement max. spacing (in) Type 1	3 Type 3	4 As (in2/ft)) Type 2
0	0.00	0.00	0.000
5	0.00	0.00	0.000

Type 1 : Smooth Wire or Plain Bars
Type 2 : Welded Smooth Wire Fabric
Type 3 : Welded Deformed Wire Fabric

Table 4 : Radial and Diagonal Tension Reinforcement

1 Section No.	2 Asmaxr (in2/ft)	3 Vb (kips/ft)	4 Vc (kips/ft)	5 Avr/s (in/ft)	6 Avs/s (in/ft)
0	1.746	16.986	13.547	0.000	0.000
5	1.746	18.540	3.230	0.000	0.000

*** P R O G R A M   P I P E - D E S I G N ***

Design Example 5.3 Oblong Pipe, Soil-Structure-Interaction, k=.1 (k/in3)

CROSS SECTION :

Pipe Width	s =	216.00	(in)
Segment Length	b =	12.00	(in)
Thickness	t =	24.00	(in)
Cross Section	Ab =	288.00	(in2)
Steel Ratio	roh =	0.40	(%)
Clear Cover	tb =	1.00	(in)
CL Steel Cover	dc =	1.50	(in)
all. Crack Width	crw =	0.010	(in)

SAFETY FACTORS :

Reduction Factor	phif =	0.90	(/)
	phiv =	0.85	(/)
Load Factor	rlo =	1.30	(/)

MATERIAL PROPERTIES :

Compression Strength	fc =	4.00	(ksi)
Yield Strength Steel	fy =	60.00	(ksi)

> D E S I G N - R E S U L T S <

Table 1 : Section Features

1 Section No.	2 Mu (k-in/ft)	3 Nu (kips/ft)	4 Vu (kips/ft)	5 req. Steel As (in2/ft)	6 Ratio roh (%)
0	+433.545	-8.138	+0.000	+0.886	+0.308
5	-319.800	-30.680	+2.275	+0.886	+0.308

Table 2 : Reinforcement for Flexural Strength

1 Section No.	2 As (in2/ft)	3 Asmin Inside	4 Asmin Outside	5 Asmaxc Compres.
0	0.283	0.886	0.665	5.050
5	0.000	0.665	0.886	4.769

Table 3 : Crack Control Design

1 Section No.	2 Type of Reinforcement max. spacing (in) Type 1	3 max. spacing (in) Type 3	4 As (in2/ft)) Type 2
0	0.00	0.00	0.000
5	0.00	0.00	0.000

Type 1 : Smooth Wire or Plain Bars
Type 2 : Welded Smooth Wire Fabric
Type 3 : Welded Deformed Wire Fabric

Table 4 : Radial and Diagonal Tension Reinforcement

1 Section No.	2 Asmaxr (in2/ft)	3 Vb (kips/ft)	4 Vc (kips/ft)	5 Avr/s (in/ft)	6 Avs/s (in/ft)
0	1.644	29.552	29.030	0.000	0.000
5	1.644	36.090	17.288	0.000	0.000

```
*****************************************
* P R O G R A M   P I P E - D E S I G N *
*****************************************
```

Design Example 5.3 Oblong Pipe, Soil-Structure-Interaction, k=.1 (k/in3)

CROSS SECTION :

```
Pipe Width        s  =    216.00 (in)
Segment Length    b  =     12.00 (in)
Thickness         t  =     24.00 (in)
Cross Section     Ab =    288.00 (in2)

Steel Ratio       roh  =     0.31 (%)
Clear Cover        tb  =     1.00 (in)
CL Steel Cover     dc  =     1.50 (in)
all. Crack Width  crw  =    0.010  (in)
```

SAFETY FACTORS :

```
Reduction Factor phif =     0.90 (/)
                 phiv =     0.85 (/)
Load Factor       rlo =     1.30 (/)
```

MATERIAL PROPERTIES :

```
Compression Strength  fc =     4.00 (ksi)
Yield Strength Steel  fy =    60.00 (ksi)
```

> D E S I G N - R E S U L T S <

Table 1 : Section Features

1 Section No.	2 Mu (k-in/ft)	3 Nu (kips/ft)	4 Vu (kips/ft)	5 req. Steel As (in2/ft)	6 Ratio roh (%)
0	+431.357	-8.177	+0.000	+0.886	+0.308
5	-318.500	-30.680	+2.249	+0.886	+0.308

Table 2 : Reinforcement for Flexural Strength

1 Section No.	2 As (in2/ft)	3 Asmin Inside	4 Asmin Outside	5 Asmaxc Compres.
0	0.281	0.886	0.665	5.050
5	0.000	0.665	0.886	4.769

Table 3 : Crack Control Design

1 Section No.	2 Type of Reinforcement max. spacing (in) Type 1	3 max. spacing (in) Type 3	4 As (in2/ft) Type 2
0	0.00	0.00	0.000
5	0.00	0.00	0.000

Type 1 : Smooth Wire or Plain Bars
Type 2 : Welded Smooth Wire Fabric
Type 3 : Welded Deformed Wire Fabric

Table 4 : Radial and Diagonal Tension Reinforcement

1 Section No.	2 Asmaxr (in2/ft)	3 Vb (kips/ft)	4 Vc (kips/ft)	5 Avr/s (in/ft)	6 Avs/s (in/ft)
0	1.644	29.552	29.030	0.000	0.000
5	1.644	36.090	17.176	0.000	0.000

* P R O G R A M   P I P E - D E S I G N *

Design Example 5.4   Oblong Pipe, Soil-Structure-Interaction, k=.1 [k/in3]

CROSS SECTION :

Pipe Width	s =	216.00 [in]
Segment Length	b =	12.00 [in]
Thickness	t =	36.00 [in]
Cross Section	Ab =	432.00 [in2]
Steel Ratio	roh =	0.40 [%]
Clear Cover	tb =	1.00 [in]
CL Steel Cover	dc =	1.50 [in]
all. Crack Width	crw =	0.010 [in]

SAFETY FACTORS :

Reduction Factor	phif =	0.90 [/]
	phiv =	0.85 [/]
Load Factor	rlo =	1.30 [/]

MATERIAL PROPERTIES :

Compression Strength	fc =	4.00 [ksi]
Yield Strength Steel	fy =	60.00 [ksi]

>D E S I G N - R E S U L T S<

Table 1 : Section Features

1 Section No.	2 Mu [k-in/ft]	3 Nu [kips/ft]	4 Vu [kips/ft]	5 req. Steel As [in2/ft]	6 Ratio roh [%]
0	+572.390	-6.773	+0.000	+0.977	+0.226
6	-409.578	-35.399	-3.172	+0.977	+0.226
15	+507.390	-40.300	-7.800	+0.977	+0.226

Table 2 : Reinforcement for Flexural Strength

1 Section No.	2 As [in2/ft]	3 Asmin Inside	4 Asmin Outside	5 Asmaxc Compres.
0	0.243	0.977	0.733	7.815
6	0.000	0.733	0.977	7.457
15	0.000	0.977	0.733	7.396

Table 3 : Crack Control Design

1 Section No.	2 Type of Reinforcement max. spacing [in] Type 1	3 max. spacing [in] Type 3	4 As [in2/ft] Type 2
0	0.00	0.00	0.000
6	0.00	0.00	0.000
15	0.00	0.00	0.000

Type 1 : Smooth Wire or Plain Bars
Type 2 : Welded Smooth Wire Fabric
Type 3 : Welded Deformed Wire Fabric

Table 4 : Radial and Diagonal Tension Reinforcement

1 Section No.	2 Asmaxr [in2/ft]	3 Vb [kips/ft]	4 Vc [kips/ft]	5 Avr/s [in/ft]	6 Avs/s [in/ft]
0	1.543	41.785	44.512	0.000	0.000
6	1.543	58.050	43.759	0.000	0.000
15	1.543	41.785	44.512	0.000	0.000

*** P R O G R A M   P I P E - D E S I G N ***

Design Example 5.4 Oblong Pipe, Soil-Structure-Interaction, k=.1 (k/in3)

CROSS SECTION :

Pipe Width	s =	216.00	[in]
Segment Length	b =	12.00	[in]
Thickness	t =	36.00	[in]
Cross Section	Ab =	432.00	[in2]
Steel Ratio	roh =	0.23	[%]
Clear Cover	tb =	1.00	[in]
CL Steel Cover	dc =	1.50	[in]
all. Crack Width	crw =	0.010	[in]

SAFETY FACTORS :

Reduction Factor	phif =	0.90	[/]
	phiv =	0.85	[/]
Load Factor	rlo =	1.30	[/]

MATERIAL PROPERTIES :

Compression Strength	fc =	4.00	[ksi]
Yield Strength Steel	fy =	60.00	[ksi]

>D E S I G N - R E S U L T S<

Table 1 : Section Features

1 Section No.	2 Mu [k-in/ft]	3 Nu [kips/ft]	4 Vu [kips/ft]	5 req. Steel As [in2/ft]	6 Ratio roh [%]
0	+569.845	-6.812	+0.000	+0.977	+0.226
6	-407.137	-35.402	-3.219	+0.977	+0.226
15	+503.925	-40.355	-7.765	+0.977	+0.226

Table 2 : Reinforcement for Flexural Strength

1 Section No.	2 As [in2/ft]	3 Asmin Inside	4 Asmin Outside	5 Asmaxc Compres.
0	0.243	0.977	0.733	7.815
6	0.000	0.733	0.977	7.457
15	0.000	0.977	0.733	7.395

Table 3 : Crack Control Design

1 Section No.	2 Type of Reinforcement max. spacing [in] Type 1	3 Type 3	4 As [in2/ft]] Type 2
0	0.00	0.00	0.000
6	0.00	0.00	0.000
15	0.00	0.00	0.000

Type 1 : Smooth Wire or Plain Bars
Type 2 : Welded Smooth Wire Fabric
Type 3 : Welded Deformed Wire Fabric

Table 4 : Radial and Diagonal Tension Reinforcement

1 Section No.	2 Asmaxr [in2/ft]	3 Vb [kips/ft]	4 Vc [kips/ft]	5 Avr/s [in/ft]	6 Avs/s [in/ft]
0	1.543	41.785	44.512	0.000	0.000
6	1.543	58.050	43.702	0.000	0.000
15	1.543	41.785	44.512	0.000	0.000

****

ROGRAM PIPE-DESIGN *

****

Design Example 5.5 Oval Conduit, Soil-Structure Interaction, k = .1 (k/in3)

CROSS SECTION :

Pipe Width	s =	216.00	(in)
Segment Length	b =	12.00	(in)
Thickness	t =	12.00	(in)
Cross Section	Ab =	144.00	(in2)
Steel Ratio	roh =	0.40	(%)
Clear Cover	tb =	1.00	(in)
CL Steel Cover	dc =	1.50	(in)
all. Crack Width	crw =	0.010	(in)

SAFETY FACTORS :

Reduction Factor	phif =	0.90	(/)
	phiv =	0.85	(/)
Load Factor	rlo =	1.30	(/)

MATERIAL PROPERTIES :

Compression Strength	fc =	4.00	(ksi)
Yield Strength Steel	fy =	60.00	(ksi)

> D E S I G N - R E S U L T S <

Table 1 : Section Features

1 Section No.	2 Mu (k-in/ft)	3 Nu (kips/ft)	4 Vu (kips/ft)	5 req. Steel As (in2/ft)	6 Ratio roh (%)
0	+186.049	-9.461	+0.000	+0.800	+0.555
7	-183.225	-27.390	+3.099	+0.800	+0.555

Table 2 : Reinforcement for Flexural Strength

1 Section No.	2 As (in2/ft)	3 Asmin Inside	4 Asmin Outside	5 Asmaxc Compress.
0	0.240	0.800	0.600	2.286
7	0.054	0.600	0.800	2.062

Table 3 : Crack Control Design

1 Section No.	2 max. spacing (in) Type 1	3 Type of Reinforcement max. spacing (in) Type 3	4 As (in2/ft) Type 2
0	0.00	0.00	0.000
7	0.00	0.00	0.000

Type 1 : Smooth Wire or Plain Bars
Type 2 : Welded Smooth Wire Fabric
Type 3 : Welded Deformed Wire Fabric

Table 4 : Radial and Diagonal Tension Reinforcement

1 Section No.	2 Asmaxr (in2/ft)	3 Vb (kips/ft)	4 Vc (kips/ft)	5 Avr/s (in/ft)	6 Avs/s (in/ft)
0	1.746	16.986	13.547	0.000	0.000
7	1.746	18.540	9.727	0.000	0.000

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* P R O G R A M   P I P E - D E S I G N *
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Design Example 5.5 Oval Conduit, Soil-Structure Interaction, k = .1 [k/in3]

CROSS SECTION :

Pipe Width	s =	216.00	[in]
Segment Length	b =	12.00	[in]
Thickness	t =	12.00	[in]
Cross Section	Ab =	144.00	[in2]
Steel Ratio	roh =	0.56	[%]
Clear Cover	tb =	1.00	[in]
CL Steel Cover	dc =	1.50	[in]
all. Crack Width	crw =	0.010	[in]

SAFETY FACTORS :

Reduction Factor	phif =	0.90	[/]
	phiv =	0.85	[/]
Load Factor	rlo =	1.30	[/]

MATERIAL PROPERTIES :

Compression Strength	fc =	4.00	[ksi]
Yield Strength Steel	fy =	60.00	[ksi]

> D E S I G N - R E S U L T S <

Table 1 : Section Features

1 Section No.	2 Mu [k-in/ft]	3 Nu [kips/ft]	4 Vu [kips/ft]	5 req. Steel As [in2/ft]	6 Ratio roh [%]
0	+187.175	-9.439	+0.000	+0.800	+0.555
7	-184.259	-27.382	+3.120	+0.800	+0.555

Table 2 : Reinforcement for Flexural Strength

1 Section No.	2 As [in2/ft]	3 Asmin Inside	4 Asmin Outside	5 Asmaxc Compres.
0	0.243	0.800	0.600	2.286
7	0.056	0.600	0.800	2.062

Table 3 : Crack Control Design

1 Section No.	2 Type of Reinforcement max. spacing [in] Type 1	3 Type 3	4 As [in2/ft]] Type 2
0	0.00	0.00	0.000
7	0.00	0.00	0.000

Type 1 : Smooth Wire or Plain Bars
Type 2 : Welded Smooth Wire Fabric
Type 3 : Welded Deformed Wire Fabric

Table 4 : Radial and Diagonal Tension Reinforcement

1 Section No	2 Asmaxr [in2/ft]	3 Vb [kips/ft]	4 Vc [kips/ft]	5 Avr/s [in/ft]	6 Avs/s [in/ft]
0	1.746	16.986	13.547	0.000	0.000
7	1.746	18.540	9.736	0.000	0.000