UMTA-MA-06-0100-83-3
DOT-TSC-UMTA-83-16

U.S. Department of Transportation

Urban Mass Transportation Administration

Design Recommendations for Concrete Tunnel Linings

Volume II: Summary of Research and Proposed Recommendations

S.L. Paul
A.J. Hendron
E.J. Cording
G.E. Sgouros
P.K. Saha

University of Illinois at Urbana-Champaign
Department of Civil Engineering
Urbana IL 61801

November 1983
Final Report

UMTA Technical Assistance Program

NOTICE

This document is disseminated under the sponsorship of the Department of Transportation in the interest of information exchange. The United States Government assumes no liability for its contents or use thereof.

NOTICE

The United States Government does not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
UMTA-MA-06-0100-83-3		PB84 158047

4. Title and Subtitle	
Design Recommendations for Concrete Tunnel Linings. Volume II: Summary of Research and Proposed Recommendations.	5. Report Date: November 1983
	6. Performing Organization Code: DTS-77
7. Author(s): S. L. Paul, A. J. Hendron, E. J. Cording; G. E. Sgouros, and P. K. Saha	8. Performing Organization Report No.: DOT-TSC-UMTA-83-16
9. Performing Organization Name and Address: University of Illinois at Urbana-Champaign* Department of Civil Engineering Urbana, Illinois 61801	10. Work Unit No. (TRAIS): MA-06-0100(UM476/R4642)
	11. Contract or Grant No.: DOT-TSC-1504
12. Sponsoring Agency Name and Address: **U.S. Department of Transportation Urban Mass Transportation Administration** 400 Seventh Street, S.W. Washington, D. C. 20590	13. Type of Report and Period Covered: Final Report April 1978-December 1982
	14. Sponsoring Agency Code: URT-10

15. Supplementary Notes

*Under contract to: U.S. Department of Transportation
Research and Special Programs Administration
Transportatinn Systems Center
Cambridge, Massachusetts 02142

16. Abstract

The overall objective of this research effort was to formulate recommendations for the structural design of final concrete linings for tunnels and underground stations for mass transit use that are based on ultimate strength concepts of concrete behavior and that take full advantage of interaction between the linings and ground. The report describing this work is in two volumes. Volume I contains a detailed description of the test arrangements, results, and numerical studies.

This report, Volume II, presents design recommendations for concrete tunnel linings for transportation tunnels. The recommendations developed as a result of in-depth analysis and model testing of the behavior of concrete tunnel linings. The research addressed problem areas in current design practice, and the authors point out that the results have provided insight into the areas of uncertainty that have led designers to gross overconservatism in tunnel lining design.

The recommended procedures take into account ultimate strength behavior of reinforced and unreinforced concrete linings and provide sufficient latitude for designers to exercise judgement gained through experience and allow the flexibility required by site-specific conditions. Details of the suggested approach are based on procedures that have been accepted for years in the design of above-ground structures, with appropriate modifications to capitalize on the benefits of ground/structure interaction.

17. Key Words	18. Distribution Statement
Concrete Tunnel Linings; Ground/Structure Interaction; Tunnel Design; Tunnel Linings; Ultimate Strength Design	**Available to the Public through the National Technical Information Service, Springfield, Virginia 22161.**

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	176	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

PREFACE

The studies described in this report were performed by the staff of the Department of Civil Engineering of the University of Illinois at Urbana-Champaign, Urbana, Illinois during the period April 1978 to April 1983. The work was sponsored by the U.S. Department of Transportation, Urban Mass Transportation Administration under contract with the Transportation Systems Center, Cambridge, Massachusetts and performed under the technical direction of Mr. Gerald Saulnier and Ms. Beth Madnick. Their helpful suggestions, advice and patience are gratefully acknowledged.

This organization and the technical monitors organized a review of the research and recommendations in June 1982 that was attended by many members of the tunnel design community. This meeting was extremely helpful in further assessment of the recommendations and evaluating reaction by designers. For these efforts, we are grateful to the organizers and to the many engineers who took the time to review the report and attend the meetings.

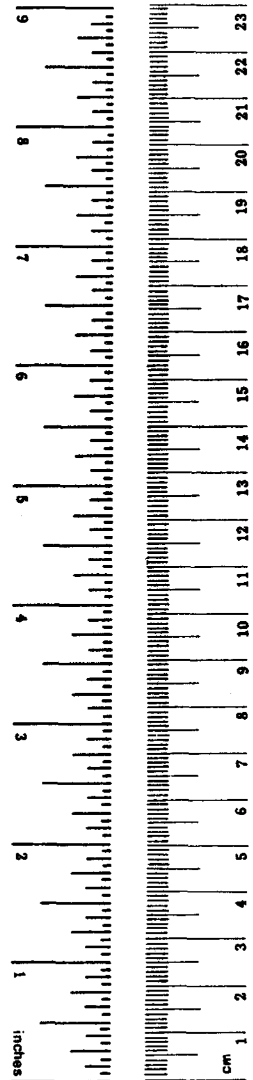
During the project, a review of tunnel design practice was conducted by Dr. George Sgouros by interviewing members of several design firms and contractors. We wish to acknowledge the assistance of the tunnel design and construction community and thank those who participated for their time and assistance.

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F

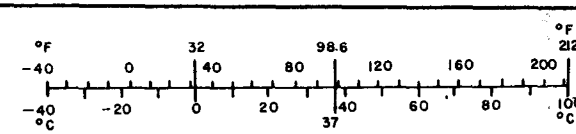


TABLE OF CONTENTS

Page

LIST OF FIGURES

FIGURE | Page

FIGURE		Page

LIST OF TABLES

LIST OF SYMBOLS

A = area of lining/unit length

A_s = area of the lining reinforcement/unit length

B = width of the tunnel opening

b = length of lining under consideration in the longitudinal direction

C = compressibility ratio for the lining-medium system

C_o = arc length of the arch that is under compressive load from the footing to the point where separation of arch and medium occurs (normally about one-third the lining diameter)

C_1 = width of the footing of an arch-shaped lining

c = cohesion of the medium material represented by the interface element between the lining and soil

D = diameter of a circular tunnel lining

E_ℓ = modulus of elasticity of the lining

E_m = in situ modulus of elasticity of the medium

e_ℓ = eccentricity of thrust in a concrete section from a linear analysis

F = flexibility ratio for the lining-medium system

f'_c = compressive strength of the concrete

f_y = yield strength of the lining reinforcement

G = shear modulus

H = depth of the tunnel from the surface

H_T = height of the tunnel opening

I = moment of inertia/unit length of lining

K_F = stiffness of the spring that represents the footing of an arch-shaped lining

K_o = lateral pressure coefficient in the medium

K_r = radial spring stiffness representing the medium

K_t = tangential spring stiffness representing the shear stress between the lining and medium

k = modulus of subgrade reaction

P = total load applied to the lining

p = pressure applied to the lining in the gravity loading analysis

p_a = air pressure used inside a tunnel

R = mean radius of the lining

T_a = thrust at a critical section in the lining due to the applied active ground pressure

T_o = ultimate thrust capacity if there is no moment acting at the section

T_u = ultimate thrust capacity of the lining at a critical section

t = thickness of the lining

u_p = displacement of the medium at the tunnel sides when the lining and medium make contact

u_t = total displacement of the medium at the tunnel sides if there is no support provided

γ = unit weight of the medium

γ_ω = unit weight of water

Θ = angle subtended by one beam element in the beam-spring model

ε_c = maximum compression strain in the concrete lining

ε_t = maximum average tension strain at the tension face of the concrete lining

ν_ℓ = Poisson's ratio of the lining

ν_m = in situ Poisson's ratio of the medium

ρ = ratio of area of reinforcement to lining area/unit length of lining

ϕ = angle of internal friction of the medium material represented by the interface element between the lining and soil

SUMMARY

The research described in this report was sponsored by the Urban Mass Transportation Administration of the U.S. Department of Transportation, and performed under the technical direction of the Transportation Systems Center, Cambridge, Massachusetts. The objective of the work was to provide recommendations for structural design of final concrete linings for tunnels and underground facilities for mass transportation use, that include ultimate strength design and ground-lining interaction concepts, and to improve the understanding of lining behavior as it interacts with the ground. The need for a set of guidelines and recommendations evolved from the perception that there has been little uniformity of approach to lining design that has sometimes led to over design and poor economy. Also, many designers have indicated a desire for a reasonable set of flexible guidelines that can form a norm of standard practice from which designers can deviate as local conditions vary.

The report describing this work is in two volumes: this one, Volume II, contains a summary of the research and the design recommendations, while Volume I contains a more detailed description of the test arrangements, results, and numerical studies.

In this volume, general recommendations are made for the design of concrete linings for tunnels in rock and soft ground with a typical diameter of 20 ft (6 m), and for large openings in rock with spans of 40 to 60 ft (12 to 18 m). A step-by-step method is not given in order to allow the designer freedom in adapting the approach to local geology, specific requirements of the supports or his own views of support behavior; however, two analysis approaches are described, one for rock and one for soft ground, that take into account interaction of the lining and ground and are adaptable to most geologic settings. The primary focus is on the structural behavior of the lining and the effect that the ground and the excavation and support process have on this behavior. Though typical loadings are discussed and their influence on the lining strength has been investigated, specific

recommendations for loading to be used are not made because they are dependent on local geology and construction procedures.

The recommendations for design are based on the same general philosophy as that used for above-ground concrete structures and described in the ACI Building Code Requirements for Reinforced Concrete, ACI (318-77). That is, the loads that are expected to act on the lining during the service life of the lining are selected and multiplied by load factors that depend on the confidence the designer has in their accuracy. These load factors are intended to make the lining strength sufficiently high that the service load stresses are low enough to prevent excessive creep, cracking, failure under accidental overload or inaccuracy in the analysis procedure. With these loads, an analysis is performed that accounts for interaction between the lining and medium and utilizes both the moment and thrust capacity of the lining. Once moments, thrusts and shears are obtained from the analysis, they are compared with the corresponding strength of the lining sections, reduced by a factor that accounts for possible variation in material strength or inaccuracy in the strength computations.

The analyses recommended for use in the design of final linings in rock and in soft ground are different because of the way loads are applied and interaction occurs between the lining and ground in the two cases. In rock, the final lining is placed in a stable opening and is initially unstressed; subsequent loading must result from rock that has loosened or relaxed and rests on the lining, and interaction results from the lateral pressure between the medium and lining as the lining deforms. Linings in soft ground are loaded by deformation of the medium and therefore the loading cannot be separated from the interaction process; also the vertical loading is accompanied in most cases by an increase in horizontal pressure in addition to the horizontal pressure resulting from deformation of the lining and the vertical loading is changed by the lining deformation.

It is recognized that a minimum constructible concrete lining is adequate in many cases, such as circular linings in competent rock or stiff soil, and that an analysis may not be necessary for these cases. However, if a verification of this idea is needed, the opening is large, the shape of the opening is not circular or the modulus of the medium is low, then the described design approach is recommended. Special loading cases, such as squeezing or swelling ground, require special considerations and are not treated specifically.

The design recommendations are based on studies that can be divided into three separate categories. These are first a survey of existing design practice in which 16 design firms and several contractors were interviewed; the second is a series of laboratory model tests of tunnel linings in various media; the third is development of a computer based finite element analysis that was used to perform parameter studies to investigate the major variables affecting the problem.

During the interviews of design firms, it was found that most designers contend that the final lining of minimum thickness for convenient construction, varying from 8 to 12 in. (200 to 300 mm), is adequate for running tunnels in rock, though some of them do analyze them for rock loads and/or water pressure loading. Large openings in rock are normally designed for rock loads that depend on the geology at the site, though there is considerable variation in the loads selected. Of nine firms who described their analysis for large openings, six of them used a series of beam elements to represent the lining, applied loads directly to the lining and represented the medium by springs or in one case by continuum elements and in another case considered the rock to be rigid.

There was greater variation in the approaches to soft ground lining design. Seven firms described the loadings used, of which four used full or partial overburden depending on the depth for the vertical load, and applied lateral pressure equal to some portion of the vertical pressure or determined the design moment by assuming an ovaling of the lining of a fixed magnitude or applied the thrust determined from the overburden at a fixed eccentricity. One firm used a vertical load of 1.5 to 2.0 tunnel diameters for initial supports and 60 percent of this value for the final lining without applying a horizontal pressure except for that resulting from ovaling of the lining during the interaction process. The remaining firms assumed that a minimum-thickness lining with a predetermined design based on experience was adequate.

Six firms described their analysis for linings in soft ground of which three used beam elements to describe the lining and radial springs for the soil, and of these, two apply a lateral pressure in addition to that resulting from ovaling of the lining while the remaining one does not, but the vertical load is limited in that case.

Thirteen firms described their strength criteria for checking the sections of the linings after the analysis is performed for both running tunnels in soil and rock and large openings in rock. Seven firms use working stress design methods, four use ultimate strength design and two use both procedures.

Also from these interviews, it appears that there are several areas of uncertainty either indicated by the variety of procedures used or statements of the designers. Among these are the use of reinforcement to prevent cracking and leakage, treatment of external water pressure, loadings for linings in soft ground, analysis and design of precast segmented linings, the degree of conservatism in large openings and treatment of initial supports in the design of final linings.

Tests were performed on models of arches that were six feet diameter, embedded in a medium to simulate rock or very firm soil, and loaded in the crown region to represent loosening loads with various shapes. The tests showed that when interlocking was present between the medium and lining, a triangular load with the peak at the crown resulted in the largest moment in the lining or lowest ratio of ultimate thrust to that at failure with pure axial thrust (T_u/T_o). When the interlocking was removed the thrust ratio was much smaller.

Stiffness of the medium was found to be the most important parameter in determining lining strength but even with the stiffest medium tested, only 62 percent of the axial thrust capacity of the lining could be utilized in resisting load. Cracking of the lining due to flexure appeared to be of little concern in these stiff media, with the first crack occurring near or above one-half the ultimate load. When the medium is sufficiently stiff, it was found that there is enough thrust to prevent tension in the lining unless the loading is concentrated or the medium is soft. Reinforcement served to distribute cracks more uniformly when they occurred, and therefore keep the individual cracks finer.

Parameter studies of arches in stiff media, with a finite element analysis that would allow nonlinear behavior of the lining, served to confirm the findings of the model tests and allowed the parameters to be varied over a much wider range. With this program, the lining behavior was investigated as medium stiffness, radius to thickness ratio of the lining and tangential stiffness between the lining and medium were varied.

Circular lining models of 44 inches diameter, embedded in a soft medium that simulated soil, were tested by applying uniform pressure to the medium surface. It was found that the medium stiffness is the most important parameter affecting lining strength, even more important than it was for arches in the stiffer medium. Cracking occurred at a much lower load relative to the ultimate because of the larger moment to thrust ratio, and reinforcement served to distribute the cracks and keep them finer. Tests on segmented lining showed loads comparable to those of their monolithic counterparts; the reduced strength of joints appeared to be compensated by the smaller moment in the linings resulting from the joints. Also the overall deformations were comparable because the medium is the dominant factor in deformation rather than the lining.

An analysis of circular linings in soft media, that included nonlinear lining behavior and interface elements to represent the shear stress between lining and medium, was performed to expand the range of parameters studied in the model tests. It was found that the angle of internal friction of the interface elements had a significant effect on lining strength. The manner in which load was assumed to reach the lining was also found to have a significant effect and is related to the construction process. External water pressure applied to the lining in addition to ground loads was shown to improve lining strength by increasing the thrust, while the moment is unaffected, resulting in a smaller moment-thrust ratio. The effect of nonlinear behavior could be studied with the analysis program, and it was found to provide considerable additional strength over that predicted by a linear analysis. The effect of placing joints in the lining as in a precast segmented system was studied and the reduced stiffness for the particular joint locations considered was determined for various medium stiffnesses.

CHAPTER 1

INTRODUCTION

1.1 OVERVIEW

The research described in this report was funded by the Urban Mass Transportation Administration of the U.S. Department of Transportation and performed under the technical direction of the Transportation Systems Center, Cambridge, Massachusetts. The realization by these and other agencies that construction costs for underground transit facilities are increasing much more rapidly than other construction costs has led to a search for ways to reduce them. Excavation of the underground opening and its support, if required, is a significant part of the total cost, and therefore substantial savings might be realized by more efficient designs of the support system. In addition, review of the design of existing linings show that there is little uniformity of approach, and though they are safe, some are more efficient than others.

These considerations have led to the research effort described in this report, that has the twofold objective: 1) make recommendations for the structural design of final concrete linings that encompass ultimate strength concepts and take full advantage of interaction of the lining with the surrounding medium, and 2) improve the understanding of lining-medium interaction and lining behavior near the failure-load range. The research is described in two volumes. This report, Volume II, contains the design recommendations and a summary of the research findings. Volume I contains the details of the model tests and parameter studies.

General recommendations are outlined in Chapter 4 for the design of final concrete linings for tunnels in rock and soft ground and for large openings in rock. A step-by-step method is not given in order to allow the designer freedom in adapting the approach to local geology or specific requirements of the supports; however, an analysis technique is described that takes into account interaction of the lining and ground and is adaptable to most geologic settings. The primary focus is the structural behavior of the lining and the effect that the medium and the excavation and support process has on this behavior. Though typical loadings are discussed, and their influence on the lining strength have been investigated, specific loadings to be used are dependent on local geology and construction techniques used. To assure that the ideas suggested for the structural design of linings are reasonable and do not conflict with other practical design and construction requirements, many discussions have been held with designers of various types of underground supports.

Special loading cases, such as squeezing and swelling ground, require special consideration and are not treated here specifically; however, many of the conclusions and analysis procedures developed would be applicable to these cases with proper modifications of the loading conditions.

The philosophy of the recommended design approach is similar to that of "Building Code Requirements for Reinforced Concrete" (ACI 318-77). This design method consists of selecting loads that are actually expected to occur and multiplying them by load factors; an analysis of the system is then performed with the factored loads acting on the structure; the results of the analysis are compared with the lining strength that has been modified by capacity reduction factors to account for uncertainties in strength. The safety factor results from combining the load factors and capacity reduction factors. Specific recommendations unique to lining design are made for the load factors to be applied and the analysis method to be used.

The analysis procedure suggested for openings in rock and for gravity loading in soil is based on representing the lining by a series of beam elements and the medium by a series of radial and tangential springs and performing a linear frame analysis. The loads may be assumed to occur from loosened rock blocks resting on the lining or a mass of soil above the crown that relaxes and rests on the lining because of high shear stresses in the soil. This analysis is adaptable to variable geology, shapes, loadings, tangential stress conditions at the lining-medium interfaces, joints in the lining, cracking of the lining and variable lining cross section; even more important, this analysis contains the important lining-medium interaction components that occur in the ground for this type of loading and the construction procedure normally used.

For some other soil conditons or a soft homogeneous rock, where loads on the lining result from deformation of the medium, the loading cannot be separated from the interaction process and there are interaction components that are active and that are not included in the analysis described above. For example, the actual load on the lining is not known because part of it is arched around the lining during the interaction. Therefore, it is necessary to consider the ground to be a continuum around the lining in order to obtain a realistic analysis. If the lining-medium system can be approximated by a uniform circular lining with linear behavior in a homogeneous linear medium, then a closed form solution is available (Ranken, Ghaboussi and Hendron, 1978). In many cases, this solution will suffice. The loading is the in situ medium stresses at the tunnel location. If a more detailed analysis is required or if the approximation described above cannot be made, then it is appropriate to represent the medium by continuum elements in a finite element analysis. In a soft medium where horizontal bedding planes or discontinuities may occur (i.e., stiff fissured clays) or where the cohesion may be low (i.e., loose sands), soil may rest directly on E the lining as described above for jointed rock. If this condition is suspected to occur, the lining should be checked for this loading and a correction procedure is provided in Chapter 4 to obtain the results of a nonlinear analysis by performing a linear analysis.

It is recognized that a minimum constructible lining is adequate in many cases such as for circular linings with a diameter up to 20 ft (6.0 m) or perhaps larger in competent rock or stiff soil, and that an analysis may not be necessary for these cases. However, if a verification of this is required, the opening is larger or the medium is very soft, then the suggested analyses may be used. Performance of analyses such as those described above require certain lining and medium properties that may be difficult to obtain with accuracy or may be quite variable. Normally, however, reasonable upper and lower limits can be placed on these properties and the analysis can be used to predict behavior under normal as well as the most unfavorable combinations of conditions; it may still be found that a minimum lining is adequate even under the most unfavorable conditions, and so no further analysis is necessary; if it is found that problems may occur if these conditions exist, a more careful investigation must be performed.

The recommendations in Chapter 4 are based on a series of model tests, parameter studies using a finite element analysis, a series of interviews with tunnel designers and contractors, and a survey of the literature. The model tests, described in Section 3.1, served to identify overall lining behavior, modes of failure, and provide strength and cracking information. They also served to verify the analysis described in Section 3.2. This analysis represents the lining with a series of beam elements that may have linear or nonlinear properties. Geometric nonlinearity is also taken into account in the solution process. The medium can be represented by continuum elements or by radial and tangential springs. Special elements may be used between the lining and medium when the medium is represented by continuum elements to model various conditions of slip at the lining-medium interface.

Seventeen firms that are involved in the design of underground supports of various types were interviewed in order to determine what procedures are now being used and what these firms feel are the most urgent problems in design. These interviews were also helpful in determining how recommendations would be most consistent with existing practice and therefore would be most likely to be adopted for future designs. The results, of these interviews are summarized in Chapter 2.

The remainder of this chapter is devoted to a discussion of the background and thinking that has gone into the recommendations proposed.

1.2 GROUND BEHAVIOR

1.2.1 Loading on Concrete Linings

Support in tunnels is used in order to (1) stabilize the tunnel heading and protect the men and equipment operating in the tunnel, (2) minimize ground movements that can damage structures and utilities, and (3) permit the tunnel to perform its intended function over the life of the project. Traditionally, the first two functions are provided by an initial support system, whereas the third function is provided by a final lining, usually concrete, installed at some time after the tunnel has been stabilized with the initial support. Linings which serve as both initial and final support, such as pre-cast concrete segments, have been finding increasing use on tunnel projects.

The loading on a final lining and its required capacity is very dependent on when and how it is installed and on the loadings that will occur after it is installed. Often, a final lining installed after the tunnel has been stabilized by the initial support will undergo very little additional loading.

The permanent concrete lining, if installed after ground loads have equilibrated, will be subjected to the loadings due to its own installation, such as the pressures applied by contact grouting and stresses due to thermal effects, and by any subsequent changes in compressed air pressure, ground water pressure, time-dependent soil or rock creep, nearby excavation or superposition of loads, such as fill.

Outline of Loading Conditions

Some of the loading conditions that can affect a final concrete lining are summarized below:

a) If the concrete lining is placed near the advancing heading, it will essentially serve as part of the initial support. Loads can develop due to the tendency of the ground to displace inward around the tunnel heading either due to elastic deflection, squeezing or loosening of the ground. The resulting loads are what the initial support system would be expected to carry.

b) Stresses generated by shrinkage and temperature changes in the concrete as it sets.

c) Grouting pressures (principally due to contact grouting behind the lining).

d) Changes in water pressure are likely to occur after the tunnel lining is installed if the drainage system clogs, the dewatering wells are turned off, or the natural drainage into the tunnel is reduced by the presence of the lining.

e) Removal of air pressure after the lining is installed.

f) Additional pressure will be applied to the lining due to changes in ground stresses if a second tunnel passes the lined tunnel.

g) Continued creep, squeeze or swell of ground surrounding the tunnel will apply additional pressure to the lining. These conditions occur principally in shales, clays or other materials which have significant time-dependent behavior.

h) Loss of lateral support of the lining due to adjacent excavation.

i) Surface loads applied after the lining is in place. Fill placed over a tunnel in soft compressible soils can cause large increases in loads on a tunnel lining.

j) Transfer of load from the initial support system to the final lining, either due to creep or deterioration of the initial lining.

Additional Loads for Precast Segmented Lining

a) Handling stresses prior to installation.
b) Thrust from the shield if segments are used as a reaction for jacking the shield forward.
c) Pressures due to expansion or grouting of concrete segments to fill the void between soil or rock and the segment.
d) Nonuniform loads and distortions due to incomplete grouting or misalignment of segments.

1.2.2 Ground Loads: General Considerations

If a tunnel were constructed in such a way that a rigid lining could be installed with no inward movement, the ground pressures acting on the lining would be the same as the initial stresses existing in the ground before the tunnel was excavated. If inward ground movements are permitted, either due to the presence of a non-rigid lining, or due to delay in placing the lining, then the pressure applied to the tunnel lining would be reduced below the level of the original in-situ stress.

With sufficient inward movement, which may be large or small depending on the stiffness of the soil or rock, ground loads on the lining will reduce to a value that is related to the pressure produced by the weight of soil or rock immediately around the opening; thus the value is proportional to the size of the opening and is an inverse function of the strength of the soil or rock medium.

In ground that tends to creep, loads may build up with time, to a value that is a function of the overburden pressure, the restraint provided by the lining, and the creep charateristics of the material. Thus, the ground loads that can develop on the tunnel lining can range widely, from full overburden pressure, a fairly concentrated load applied by a small rock slab, or even to no load for a self-supporting rock or a lining installed after all ground movements have taken place.

The following sections provide background information on the loadings that can develop for the various ground and lining conditions. Both the magnitude of the pressure and its distribution around the tunnel are of concern, as these affect not only the levels of thrust but also the bending that develops in the lining. The thrusts and moments that develop in the lining will be principally a function of the initial load distribution and the flexibility ratio, a measure of the relative stiffness of the lining with respect to the soil or rock medium.

1.2.3 Behavior of Linings in Soil

In soils, the tunnel heading is temporarily supported by a shield with the initial lining installed in the tail of the shield or the initial support is placed as close to the heading as possible. Subsequently, one of two courses is normally followed. If the initial support is concrete segments, it may also be used as the final lining and no additional lining is added; if the initial support is steel ribs and timber lagging or steel liner plate, it may only serve to stabilize the opening until a final concrete lining can be placed after the mining operation is completed. To understand the forces acting on the initial and final linings and see how they should be designed, it is first necessary to understand the construction process and what has occurred in the ground during this process.

As the tunnel heading progresses, overburden stresses tend to push the face of the excavation inward, so there is both radial and longitudinal movement directly at the face. If no supports were placed to restrain the ground, the radial movement would increase with distance behind the face until a constant deformation is reached two to three tunnel diameters back. The deformation will also increase with time due to creep, the amount of increase depending on the type of soil. In soft clay, the increase with time will be appreciable while with sandy soils it may be small. The longitudinal distribution of radial displacement is shown for a hypothetical case in Figure 1.1,

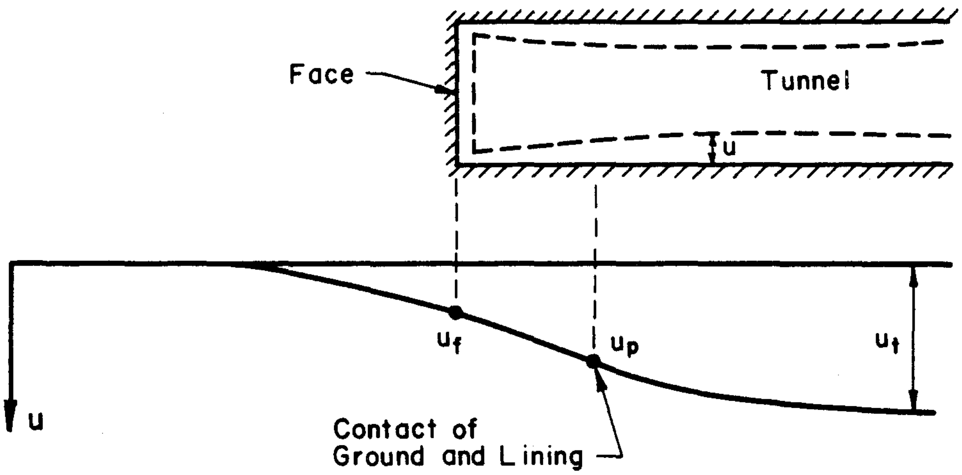


FIGURE 1.1 LONGITUDINAL DISTRIBUTION OF DISPLACEMENT NEAR THE TUNNEL FACE (FROM RANKEN, GHABOUSSI, HENDRON, 1978)

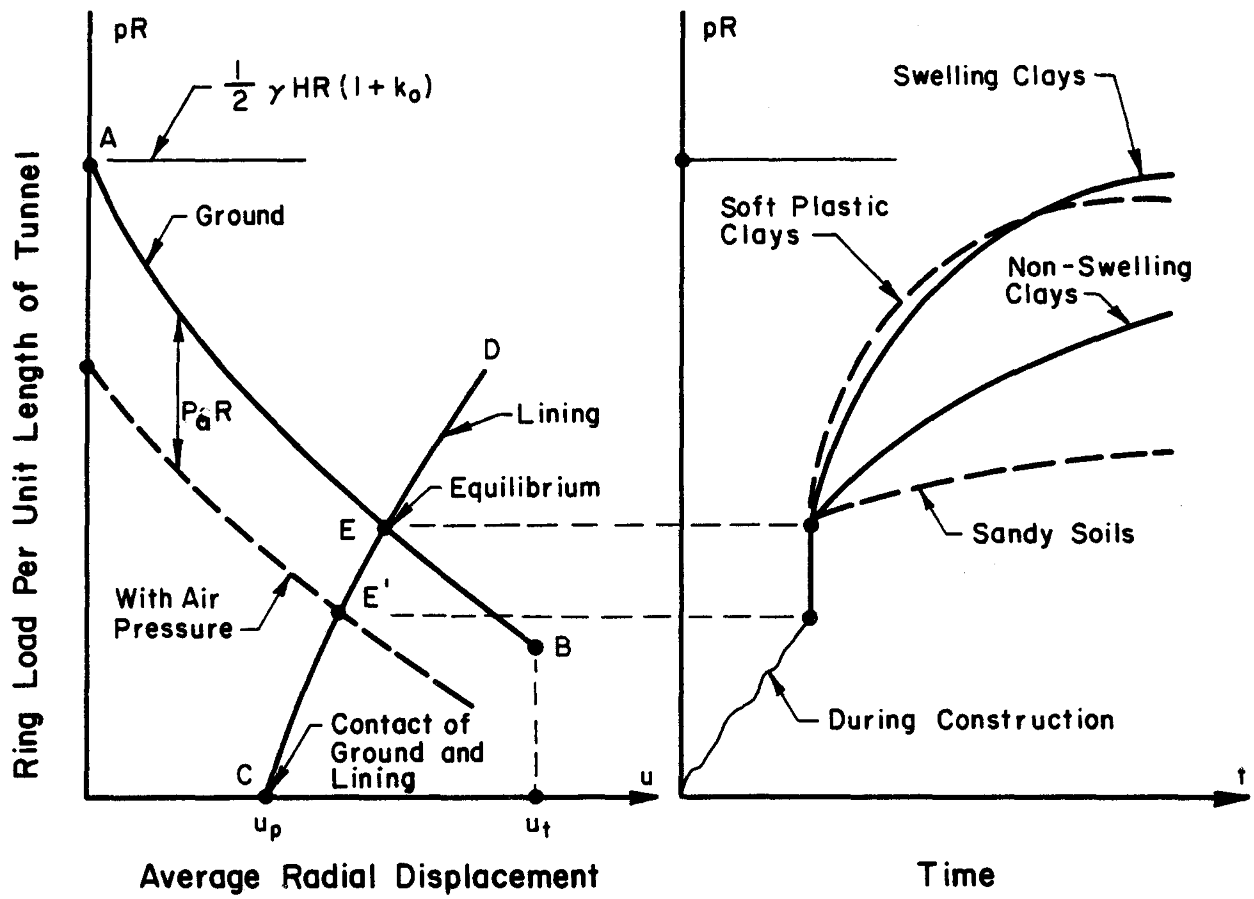


FIGURE 1.2 HOOP LOAD INTERACTION BETWEEN THE GROUND AND LINING AND LOAD-TIME CURVE FOR THE LINING (FROM PECK, 1969)

taken from Ranken, Ghaboussi and Hendron (1978). Interaction will clearly depend on when the lining is placed and how much ground movement has occurred when contact is made between the lining and ground. That is, if the lining is placed when the displacement u_p has occurred in Figure 1.1, the interaction between the soil and initial lining will depend on the ratio of u_p/u_t where u_t is the total displacement if no support was provided. The displacement u_p depends on the distance behind the face that the lining is placed and the void between the ground and the lining. A void is usually left between the ground and initial lining, and an attempt is made to either expand the lining to eliminate it or to fill it with pea gravel and/or grout soon after the support is placed. The success of this attempt depends on the rate of movement of the soil and the care exercised during construction.

The hypothetical ground reaction curve for ring thrust shown in Figure 1.2 is useful to show how the lining and ground interact, (Peck, 1969). The average ring load, if no radial deformation occurs, will be approximately the lining radius times the mean of the vertical and horizontal ground pressures; if the vertical pressure is given by γ H and the horizontal by KγH, then the maximum ring thrust at point A is $1/2\ H(1 + K_o)\ \gamma R$, where R is the outside radius of the unlined opening and γ the unit weight of soil. If the boundary of the opening is allowed to displace inward, the corresponding ring thrust required to prevent further displacement decreases along curve AB. The shape and position of this curve depends on the stress-strain-time behavior of the soil and results from a change in the stress pattern around the opening that allows some load to arch around it and therefore not be applied to the lining. If pressure were applied to the outside of the lining after the deformation u_p has occurred, the lining would deform along line CD. At the point E, where the two curves intersect, the internal pressure on the medium and the external pressure on the lining are in equilibrium.

As displacement is allowed, the lining pressure reduces to a value which is a function of the weight of the material that must be supported immediately around the opening and is thus proportional to γR, the unit weight of the soil times the radius of the opening. In frictional materials, the minimum value of load can be expressed as γ R divided by a frictional coefficient, $f(\phi)$. This is the gravity load conditions, sometimes referred to as a "loosening condition" although it can take place without cracking and separation of blocks of soil from the surrounding medium. In stiffer soils, the deformations required to achieve the minimum values are quite small and it is possible, even when the tunneling procedures are designed to prevent excessive ground deformations, to approach or achieve the minimum values. In frictional materials, such as sand, the minimum ground pressures will be equivalent to the pressures applied by a height of soil extending approximately one-half to two diameters above the tunnel crown.

In soft clays, the deformations required to achieve the minimum values are large, and pressures acting on the lining will be greater than these values. In addition, because of the time-dependent behavior of the clay, the lining pressures will tend to increase with time to values which are a function of the total overburden pressure. Thus, in many soft clays, final pressures are assumed to be close to the total overburden pressure.

An important aspect of tunnel construction is ground water control, which may be accomplished by preconstruction dewatering (pumping through wells installed at the surface) or by pressurizing the tunnel face with compressed air or slurry-face machine systems. Differences in ground water control can produce significant variations in distribution of ground stresses and in the loads applied to the lining, particularly linings constructed in stages. If the excavation is made under air pressure, the relation between ring load and displacement for the soil would be displaced downward in Figure 1.2 by the air pressure times the radius, $p_a R$; if the lining and soil make contact when the displacement is u_p, equilibrium would be reached at the point E', but the load would increase back to E when the air pressure is removed. Water pressure applied subsequent to construction of the lining would raise the curve for the ground thrust by the water pressure times the radius of the opening, $p_w R$ and result in equilibrium at a point on CD above E.

Passage of time may result in further increase in the ring loads as shown in the right side of Figure 1.2 where the amount of increase depends on the type of soil. The ring thrust may approach that for full overburden in soft plastic clays and may increase very little in a sandy soil.

Though the diagram shown in Figure 1.2 would be difficult to construct for a particular case and does not include the moment arising from nonuniform loading, it shows the important influence that the construction process has on the lining load because of its effect on u_p, which is made up of the deformation that occurs prior to lining-ground contact and includes the gap between lining and ground. It also shows that the average overburden pressure is a reasonable upper limit for determining ring thrust in the lining even when the effect of creep is included and that the thrust will almost always be less than overburden pressure times the radius. The deformation that occurs toward the lining in a soft soil before it contacts the lining is usually large in comparison with the deformation of the lining after contact is made.

In reality, linings are neither perfectly flexible or rigid. The distortions, moments and thrusts developed in such linings can be determined by considering the relative stiffness of the lining with respect to the soil, expressed by the Flexibility Ratio "F". Both elastic continuum analyses and beam-spring structural models used to approximate the lining and soil are available and have been commonly used by designers to evaluate the effect of the relative lining stiffness on the distortions and moments in the lining for various loading configurations. Such analyses have proven very useful but have major limitations when used to evaluate the ultimate capacity of concrete linings in conditions where the eccentricity of the thrust is great enough to produce tension in the concrete linings. In such cases, concrete linings, particularly when unreinforced, have capacities well in excess of those determined from the linear analyses. The results of the research program, consisting of tests of concrete lining models and analyses that include the nonlinear behavior of the concrete, have provided information that can be used in assessing the actual capacity of such linings.

The loading on an initial lining will depend on the amount of ground movement taking place before contact is made between the lining and the ground and on the pressures that develop during grouting or expansion of the lining. Since the deformation and loading depend so much on the construction process, it can only be estimated or based on calculations that provide a reasonable upper limit for design. Some initial supports, such as steel ribs and lagging or steel liner plate, are very flexible and can deflect sufficiently to resist the ground loads principally in thrust. They continue to deform as the soil pressure changes due to advance of the heading, changes in the soil stresses with time, or adjacent construction such as another tunnel. Segmented linings that may constitute both the initial and final support may be somewhat less flexible and may suffer loss of strength at the joints if excessive rotations occur there; however, lining deformation before contact with the soil can be controlled by careful grouting and use of horizontal tie rods. Greater care is likely to be exercised in the installation of initial linings that will also serve as the final lining.

In some soils, time-dependent relaxation may continue for an extended period and additional pressure may occur after the final lining is placed. This additional load is resisted by both the temporary and final linings as a composite structure. Furthermore, many designers assume that the initial supports will deteriorate so the soil pressure that was originally resisted by them (or some portion of it) would be transferred to the final lining. If this philosophy is adopted, it is the most significant source of loads on the final lining and in most cases will control its design. In this case, the load assumed to be resisted by the temporary lining is critical in the final lining design. The initial support is usually very flexible and therefore deforms so that the external pressure is nearly uniform and it is this nearly uniform pressure that would then act on the final lining.

1.2.4 Behavior of Linings in Rock

Rock excavation is performed primarily by drill and blast techniques, by road headers (boom mounted cutters) in some softer rocks, or by tunnel boring machines where long, circular tunnels are to be excavated. When drill and blast techniques are used, an irregular surface on the interior of the opening is usually created while a fairly smooth surface is created with mechanical excavation. Vibrations from the drill and blast operation are likely to loosen the remaining rock more than the tunnel boring machine.

Many tunnels are likely to be excavated and supported temporarily, with a concrete lining placed after excavation is completed; however, rock tunnels are also supported with permanent linings installed close to the face. For example, the large openings for the subway chambers in rock on the Washington Metro were supported initially by rock bolts and steel ribs shotcreted in place, with the steel ribs and shotcrete becoming part of the final lining when additional shotcrete was added to encase the ribs. Another example is the concrete segments used for both initial and final support in TBM excavated tunnels.

Loosening Ground

Rock may support itself around an opening if it contains few joints or joints that are discontinuous and irregular, so that a structural lining may not be necessary. The geologic settings of interest, then, are those in which sufficient jointing, bedding planes, faults, or weathering exists so that rock blocks and wedges will be unstable unless supported.

Most rock masses are stiff enough that very little deformation of the tunnel is required to relieve the original overburden pressures. Even when efforts are made to install support as soon as possible behind the heading, enough displacement takes place to relieve most of the overburden pressures that could act on the lining. In such conditions, the only loads that must be accommodated by the support system are those due to the weight of the immediate blocks of rock surrounding the opening that would tend to displace into the opening, less any shearing resistance developed along the boundaries of the rock blocks. These gravity loads, often termed loosening loads, are proportional to the unit weight of the rock times some dimension which is a function of the width of the opening or the size of the rock wedges, and the configuration and strength of the rock discontinuities surrounding the opening.

If excessive displacements are allowed, the rock around the opening will loosen, the interlocks and peak strength along joint surfaces will be lost, and the ultimate lining loads will increase.

One of the earliest methods for estimating rock loads is found in Tunneling with Steel Supports (Proctor & White, 1945). In this volume, Terzaghi presented a tunnel ground classification for rock using terms, such as "intact," "stratified," "massive, moderately jointed," and "blocky and seamy," that can be considered loosening ground for steel rib supports. The design rock load recommended for these ground conditions are given in terms of a height of rock that is proportional to the size of the opening. The height of rock ranged from 0 to 0.25 B for "intact" rock, to $(B + H_T)$ for massive, moderately jointed rock to (0.35 to 1.1) $(B + H_T)$ for very "blocky and seamy" rock where B is the width of the tunnel and H_T is the height. The same rock loads are not necessarily applicable to other types of linings, because the amount of loosening developed with steel ribs and the degree of conservatism in the other assumptions used in designing ribs may differ from those used with other types of lining.

Cording and Mahar (1978) also evaluate rock loads on the basis of the size of blocks or wedges that are separated by discontinuities around the opening. They recommend examining the local geology and selecting typical rock blocks and wedges that may loosen and bear on the lining based on the orientation and character of the actual discontinuities expected at the site. The size of the critical rock wedges assumed to require support will depend on the orientation of the discontinuities and the shear strength along the joints. Large, deep wedges have the potential for failing if the discontinuities bounding the wedges are planar and sheared. The same shaped wedge could not fail if it were bounded by irregular or discontinuous joints. Such joints, however, could allow separation of a thin slab or small rock wedge located near the tunnel perimeter.

Methods for estimating loads based on rock mass characteristics, such as RQD, weathering, characteristics of joints, bedding and faults, and ground water conditions have also been proposed (Barton, et al., 1974, Wickham, et al., 1974, Bieniawski, 1974).

Tangential shear stress between the lining and rock influences the lining behavior a great deal and is difficult to evaluate, but in most cases, a cast in place lining will be capable of developing significant shear along the rock-lining boundary. If the lining is cast directly against the irregular surface resulting from the drill and blast technique, it seems reasonable to assume that there would be little relative tangential deformation and no slip, although incomplete grouting and the presence of voids could reduce the effective shear stress between the lining and rock. Even with a smooth TBM bored perimeter, significant tangential shear stresses will develop. The assumption of full slip and no frictional resistance between lining and rock allows large deformation of the lining and therefore large moments occur, but this condition is unrealistic in most cases. By whatever means the frictional resistance and tangential stiffness along the contact is taken into account, it is reasonable to assume that they act not only adjacent to but also in the region where the active rock load is applied.

Loads applied by loosening rock can have different shapes, depending on the geology of the rock mass. Highly fractured layered strata may loosen and apply load uniformly over the crown region or triangular wedges may ravel onto the lining and give maximum load at the crown. Larger blocks or systems of blocks may remain relatively rigid as they rest on the lining; deflection at the crown would then reduce the load at that point and allow the block to rest primarily on each side of the crown, resulting in a pressure pattern that is minimum at the crown and increases laterally in both directions. An oblique system of joints in the rock may cause the load to be concentrated at one side of the crown region or to be applied on the side of the lining. If the geologic conditions are known well enough to draw the possible patterns of joints that may exists at the site on the tunnel cross-sections, it will be possible to estimate the shape and magnitude of loadings that are likely to occur.

Methods are available for modeling the development of gravity loads by simulating the medium with blocks separated by joints that have nonlinear behavior, and these methods are most desirable when they are available. However, this type of analysis is normally too complex for the usual application and the programs are not generally available. Therefore, it is reasonable to select a simple procedure that models the final lining with the loads from rock blocks applied directly to the lining. The full weight of rock blocks and wedges reduced by friction and/or interlocking forces will generally be conservative; if sufficient information is available to predict how the load may be reduced by prompt support with rock bolts, then an advantage may be taken of these additonal load reductions as well.

Though the initial supports have stabilized the opening by the time the final lining is placed, many designers feel that the initial support should not be considered as part of the final lining because there is little control over its placement and/or it may deteriorate with time. An exception, in some cases, is the inclusion of that part of the steel ribs that are embedded in the final lining concrete or when steel ribs are used for initial support and become part of the final lining by adding shotcrete.

This approach seems reasonable in view of the protection that the concrete offers the steel. Calculations show that a final lining of minimum thickness is adequate to support the usual loosening rock loads applied to a circular or near circular tunnel with a diameter of up to 20 ft (6 m) provided adequate account is taken of radial and tangential passive resistance. Therefore it is usually only for larger openings, for noncircular shapes, or for heavy squeezing ground that use of the temporary supports in the final lining need be considered. Straight side walls in the lining are normally used only when the rock is quite competent so that lateral rock loads are not likely to be large. In this case, water pressure is then the primary loading on the walls, if drainage is not provided.

Squeezing Ground

Squeezing ground in rock is often associated with weathered zones and fault zones in which significant amounts of clay minerals are present. Squeezing can also take place in weaker rock materials, such as shale and tuff, which often contain clay minerals as well. In squeezing ground, the overburden pressures can be relieved only by relatively large inward displacements, and loads may continue to build after the lining is installed. The rate of buildup is dependent on the creep parameters of the material and on lining stiffness. If movements are prevented, loads will be high. If movements are allowed, the loads that the lining will carry will decrease, although loads and distortions will increase if uncontrolled movements are allowed. In such cases, bending effects can be severe, particularly where there is a great variation in rock properties around the perimeter of the tunnel, such as rock blocks separated by seams and zones of soft soils and where the support is not in continuous contact with the ground.

Squeezing pressures can be estimated from previous experience and from time-dependent analyses. They will be some function of the overburden pressure as well as the creep properties of the material and the amount of displacement allowed to relieve ground pressure. For deep tunnels (greater than 500 ft deep) passing through creep-sensitive materials, such as weak tuff or clayey fault gouge, squeezing pressures as high as 30 to 50 percent of the overburden pressure have been measured. Smaller values are observed where controlled ground displacement is allowed.

In many cases, the permanent lining is not installed until the rate of inward movements has become small. Thus, the only pressure the lining will sustain will be due to any tendency for small additional creep or an increase in water pressure on the lining. Practice in some 30-ft wide openings in squeezing ground in Europe is to stabilize the tunnel with shotcrete, bolts and light steel ribs and then install a nominal, 10 in. (250 mm) concrete lining after placing a waterproofing membrane after most of the ground movements have stopped.

1.3 PROCEDURES FOR ANALYZING LINING-MEDIUM INTERACTION

1.3.1 Introduction

Both linear and nonlinear analysis of the behavior of concrete linings in soil or rock are discussed. A major part of the study was devoted to development of a nonlinear analysis of the ultimate behavior of circular and arch-shaped concrete linings in soil and rock. Nonlinear analysis of the post-peak behavior of the concrete material is required in order to evaluate the ultimate capacity of a concrete tunnel lining, particularly when bending takes place. The analysis results have been compared with the model test results, and parameter studies have been performed in order to extend the range of lining flexibilities and loading conditions.

Linear analyses are also presented and discussed in this section, principally because they are readily available to designers and have been used in the past to evaluate lining behavior. The linear analyses permit a reasonable and conservative estimate of the ultimate thrust capacity to be obtained, if eccentricities are small enough to result in thrusts and moments in the compression region of the moment-thrust interaction diagram (above the balance point). However, linear analyses may result in overdesign and use of excessive amounts of reinforcement if they are used to evaluate ultimate thrust capacity in cases where eccentricities are high enough to fall well below the balance point. Procedures have been developed and are presented in Chapter 4 that permit the designer to obtain the nonlinear thrust capacity once he has performed a linear analysis. A relationship has been developed between the linear elastic eccentricity obtained from a linear analysis and the ultimate thrust determined from the nonlinear analysis.

For the purpose of discussing the analysis techniques used for soil tunnels, it is helpful to first describe the three types of loading considered. If a region of the medium containing a lined opening is isolated and a pressure is applied to the upper surface, it is considered to be an "overpressure" loading. The lining was placed in the medium when it was unstressed and the lining and medium interact as the surface pressure is applied. Lateral stress in the medium is normally handled by applying a lateral pressure to the medium. This loading represents the vertical stress in the soil at the tunnel level due to the overburden for a deep tunnel.

In reality, the lining is never placed in an unstressed medium, but the excavation is made in the medium with the overburden stresses and corresponding deformations present. The lining is then placed in the opening after this initial deformation has occurred so that it fits perfectly and before any additional deformation occurs due to excavation of the tunnel. This is called the "excavation" loading and is different from the overpressure loading in that the lining is not subjected to the medium deformation that occurs due to the in situ stress before the opening is excavated. The moment and thrust in the lining resulting from the excavation loading are smaller than those from the overpressure loading. However, the excavation loading still ignores the deformation in the medium due to the excavation before the lining is placed and the void between the lining and medium in soft ground tunnels discussed in Section 1.2.3. In terms of the deformation due to excavation in Figures 1.1 and 1.2, this case is equivalent to placing the lining ahead of the excavation so that $u_p = 0$ and the lining fits perfectly in the opening.

In rock and in some types of soil, the weight of a mass of material above the opening can rest directly on the lining, resulting in a loading equal to the weight of some depth of material reduced by the shearing stresses at the edge of the mass. This will be referred to as a "loosening" load when it occurs in rock and a "gravity" load when it occurs in soil because the mechanism is different in the two materials.

Once contact is made between the lining and medium, interaction begins because the lining reduces the deformation of the medium and the medium reduces the deformation of the lining. For the purpose of comparing the analysis techniques, it is convenient to separate this interaction into the following four components in order to see how each technique handles them.

1) When a load is applied to the crown region of the lining, the vertical diameter shortens, causing the horizontal diameter to lengthen and thus push the springline region into the medium. The lateral medium pressure on the lining thus increases and results in a stabilizing effect on the lining because it tends to resist the ovalling. This is the interaction due to overall deformation.

2) When the lining ovals, the crown region deforms so that the crown deflects more than the adjacent regions near the crown causing a change in shape of the active pressure applied in this region. Generally, the pressure will become smaller at the crown and increase laterally as the pressure arches over the crown region locally. The gradual reduction in pressure at

the crown during deformation of the lining will reduce the final crown moment below that which would have occurred if arching at the crown had not occurred or if the pressure at the crown had been following.

3) Before the excavation is made, there is a vertical in situ stress at the sides of the proposed opening that results in an in situ horizontal stress due to the Poisson effect equal to $\nu/(1-\nu)$ times the vertical stress if the medium is assumed to be elastic or K_o times the vertical stress if the coefficient of earth pressure is known. This horizontal pressure makes the all-around pressure on the lining more uniform and therefore decreases the moments by decreasing the ovaling. This pressure is in addition to the lateral pressure described in first component listed.

4) The lining is generally less stiff overall than the medium it replaced (especially if the deformation of the medium is considered before ground-liner contact is made). Therefore, load above the lining is arched around the sides increasing the vertical stress in the springline region and reducing the load applied to the lining because the total overburden load is resisted by the lining and by arching in the medium; the division of load into the two parts depends on the relative stiffness of the lining and medium. The change in vertical stress in the springline region in the medium due to arching has an additional effect of increasing the horizontal pressure on the lining , due to the Poisson effect in this region. Thus, this positive arching has two beneficial interaction components-reduction of active vertical load on the lining and increaseof horizontal passive pressure at the springlines.

1.3.2 Closed Form Solutions

Ranken, Ghaboussi and Hendron (1978) and Einstein et. al. (1980) provide a good discussion of the history and applicability of the closed form solutions for a lining embedded in a homogeneous medium. These solutions are based on two-dimensional, plane strain, linear elasticity assumptions in which the lining is assumed to be placed deep and in contact with the ground (no gap). Early solutions by Burns and Richard (1964), Dar and Bates (1974), and Hoeg (1968) were derived for the overpressure loading described above, while solutions by Morgan (1961), Muir Wood (1975), Curtis (1976), and Ranken, Ghaboussi and Hendron (1978) were for the excavation loading. Solutions for both loading conditions are available for the full slip and no slip conditions at the lining-medium interface. The full slip solution will give conservative (high) estimates of moment. The difference between the slip and no slip conditions is not great for the overpressure and excavation loading cases, but is large for the gravity loading and loosening loading cases, particularly if vertical rather than radial active loadings are assumed to act on the lining.

These closed form solutions contain all four of the interaction components described above subject to the assumptions on which they are based. They do not, however, allow for a gap to occur between the lining and medium or for the lining to be placed at a distance behind the excavation face. If the ground and lining remain linearly elastic, the excavation loading is more reasonable than the overpressure loading because the ground stress and deformation due to the in situ stress has occurred when the lining is placed. In fact, more deformation must occur when the excavation is made and before the lining and ground can make contact so this case is an upper limit for the linear elastic assumptions. However, if the medium deformations are large before the lining and medium make contact and the shear strength of the medium is exceeded (as in a soft plastic clay), time-dependent deformations may continue for some time, relieving the stresses in the soil around the opening and applying more pressure to the lining,(see Figure 1.2). In this case, the excavation loading may no longer provide an upper limit for thrust.

Design charts that provide dimensionless moment and thrust values for various ratios of deformability of the medium to that of the lining have been published for these solutions (Ranken, Ghaboussi and Hendron, 1978). Also, they provide a theoretically correct pair of dimensionless parameters for comparing the properties of the medium to that of the lining for the linear case. These parameters, called the "compressibility" and "flexibility" (Peck, Hendron and Mohraz, 1972) greatly facilitate the design process and provide insight into how medium stiffness and lining properties affect the solution for moment and thrust.

The solution for the excavation loading and full slip between the medium and lining as given by Ranken, Ghaboussi and Hendron (1978) is as follows with the symbols and sign convention defined in Figure 1.3:

$$P_r = \frac{\gamma H}{2} \{(1 + K_o)(1 - L_f) - 3(1 - K_o)(1 - 2J_f) \cos 2\theta\}$$

$$T = \frac{\gamma HR}{2} \{(1 + K_o)(1 - L_f) + (1 - K_o)(1 - 2J_f) \cos 2\theta\}$$

$$V = -\frac{\gamma HR}{2} \{(1 - K_o)(1 - 2J_f) \sin 2\theta\}$$

$$M = \frac{\gamma HR^2}{2} \{(1 + K_o) \frac{L_f}{6F} + (1 - K_o)(1 - 2J_f) \cos 2\theta\}$$

$$L_f = \frac{(1 + 2\nu_m) C}{1 + (1 - 2\nu_m) C}$$

$$J_f = \frac{F + (1 - \nu_m)}{2F + (5 - 6\nu_m)}$$

$$C = \frac{E_m}{E\ell} (\frac{R}{t}) \cdot [\frac{(1 - \nu_\ell^2)}{(1 + \nu_m)(1 - 2\nu_m)}]$$

$$F = \frac{E_m}{E_\ell} (\frac{R}{t})^3 [\frac{2(1 - \nu_\ell^2}{(1 + \nu_m)}]$$

H = Depth of the tunnel

A = Cross-sectional area/unit length of lining

I_ℓ = Moment of inertia/unit length of lining

E_ℓ = Modulus of elasticity of the lining

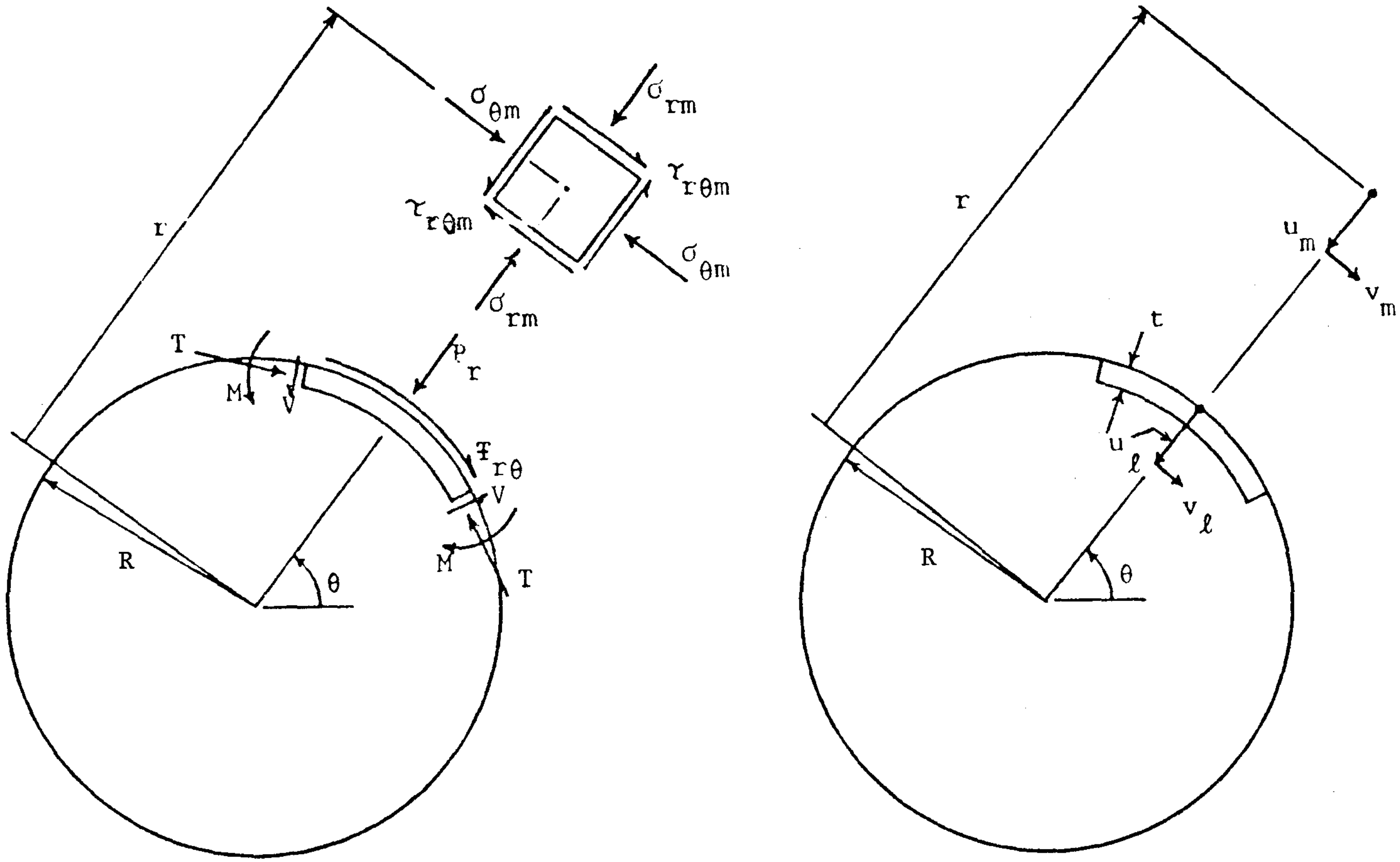


FIGURE 1.3 DEFINITION OF TERMS FOR THE EXCAVATION LOADING SOLUTION FOR INTERACTION OF A THIN LINING AND MEDIUM

In these formulas, the effective soil stresses should be used to obtain the moment, thrust and shear, and then the thrust due to water pressure added to that obtained from the formula for thrust. The water pressure is omitted from the formulas because it does not result in moment and shear.

1.3.3 Beam-Spring Model

The most important aspect of a realistic analysis is that it should allow interaction between the medium at the sides of the opening and the lining, because this stabilizing passive normal and tangential pressure improves the strength and decreases the deformation and cracking of the lining. If the lateral presence and its distribution were known at the equilibrium position, it could be applied with the loads, the lining analyzed, and a valid solution would be obtained. However, the lateral pressure depends on the properties of the lining (deformability) and the stiffness of the medium and is not known beforehand in terms of its magnitude or distribution. In the past, some design approaches have depended on a realistic estimate of this lateral pressure, have assumed it to be uniformly distributed, and have provided satisfactory designs. However, these methods do not provide for different lateral pressure when the lining stiffness changes or even when the medium stiffness changes in many cases.

An approach that would give a more realistic model of the interaction when the load is applied directly to the lining would be to replace the medium at the sides with radial and tangential springs that have the same deformation characteristics as the medium and model the lining with a series of linear beam elements. In this case, loads can be applied in any location or shape and the medium properties can be varied easily. The radial springs represent the radial passive pressure on the lining and the tangential springs represent the shear stress between the medium and lining. If the rock surface is smooth, the tangential shear stress could be small but if the overbreak is irregular as in drill and blast excavation, the shear stress will depend on positive interlock and will be high. In the analysis, radial springs that would be in tensions should not be allowed because it would imply that the medium is pulling on the lining. In the ground, the lining does not actually pull away from the medium, but there is a relief of the passive radial pressure of the ground on the lining. The tangential springs should remain active in the loaded region since the rock applying the loads would remain in contact with the lining and apply tangential stress. Examples of this model are shown in Figures 3.14 and 3.44.

There are programs available that are designed for this type of problem and automatically disconnect tension springs and therefore are convenient to use. The model can also be constructed with any frame analysis program using bar elements if spring elements are not available; it will then be necessary to disconnect radial tension elements through a trial and error process but the same solution will be obtained once the radial tension elements are made inactive. In addition, this model of behavior is suggested as a minimum analysis and is not meant to exclude more complete models of the behavior such as those that represent the ground with continuum elements and the interface between lining and medium with interface or joint elements. If these models are available, they can provide a more complete picture of behavior. The load should be applied directly to the lining in these models as suggested above, however, for the gravity or loosening load.

Loads on the final lining are likely to require a long time to reach their full value; this is particularly true if the load is resisted fully by the initial support before the final lining is installed. To account for this time effect on the deformability of the concrete lining and the consequent effect on the interaction, it is suggested that the modulus of the concrete used in the analysis to obtain the beam element properties be reduced to one-half the value given by the ACI formula (ACI 318-77). Contractors often increase the cement content of the specified mix to improve pumpability and early strength for form removal. In this case, the concrete compressive strength and liner modulus will also increase. The actual in situ liner modulus should be used if it can be determined, as it will reduce the relative stiffness of the liner and medium and increase the calculated moment. The contractor may also increase the lining thickness by making the mined excavation larger than the specified minimum. This will decrease the flexibility ratio and increase the lining moments. The effect of these variations on the design parameters should be checked by the designer.

Radial springs used to represent the medium for a circular lining may be given a stiffness based on the formula (Dixon, 1971),

$$K_r = \frac{E_m b\theta}{(1 + \nu_m)}$$

where

K_r = radial spring stiffness representing the medium included by angle ,

E_m = in situ modulus of elasticity of the rock mass,

ν_m = in situ Poisson's ratio of the medium,

b = length of lining under consideration in the longitudinal direction,

θ = angle subtended by tributary area of spring in rad.

The elastic modulus of the medium E_m for the rock mass must be reduced from the intact laboratory values of rock samples to account for the effects of joints and shear zones. The amount of reduction depends on the joint spacings, orientation, and fill material.

The following formula for modulus of subgrade reaction is suggested for use to obtain the radial spring stiffness when arches are analyzed:

$$k = \frac{E_m}{2C_o}$$

where

k = modulus of subgrade reaction,

C_o = arc length of the arch that is under compressive load from the footing to the point where separation of arch and medium occurs (normally about one-third the diameter of the lining).

The resulting subgrade modulus must then be multiplied by the tributary area for each spring. Therefore the spring stiffness would be

$$K_r = k\ R\ b\,\theta,$$

where

R = radius of the lining.

Tangential spring stiffness is more difficult to determine, but studies show that they generally lie between 20 and 50 percent of the stiffness of the radial springs depending on the surface between the lining and medium. A value near 20 percent may be appropriate when there is a smooth surface between the lining and medium while a value near 50 percent may be more appropriate for a surface with irregular overbreak. The effect of the ratio of tangential to radial spring stiffness is discussed in Chapter 3 and some guidance to the most likely value is given in Volume I of this report.

If the lining being considered is an arch with footings, then the medium deformation under the footing should be represented by a spring also. This spring should be based on the modulus of subgrade reaction and can be calculated from the formula

$$K_F = \frac{E_m}{2C_1}(C_1\ b) = \frac{E_m b}{2}$$

where

K_F = stiffness of the spring that represents the footing

C_1 = width of the footing

The number of beam elements needed to represent the lining depends on the lining geometry and the desired spacing of springs (or elements) to represent the medium. Springs should be placed at the joints at the ends of each beam element and the springs should be close enough to provide a smooth variation of tangential and radial passive resistance. If the resisting pressure has a steep variation in some region, then the beam elements should be made shorter to provide closer spacing of the springs. In general, the angle subtended by beam elements in the lining should be between approximately 7.5 and 15 degrees, with the smaller value used when practical.

Section properties for the beam elements can normally be based on the gross dimensions of the lining between node points. If the area of present steel reinforcement is small, it can be ignored in calculating the lining stiffness; if the area is large or if a composite section is used, the transformed section should be considered in calculating the section area and moment of inertia. The section dimensions should include all the concrete that can reasonably be expected to be present in the final structure. In some cases where cracking may be extensive in a small region of the lining, it would be reasonable to use a cracked or partically cracked moment of inertia for the elements in this region.

When the beam-spring model is used, additional variables can be considered that are not included in the closed-form solutions, such as a nonuniform distribution of applied load, variable lining stiffness around its circumference, joints in the lining, or variation of ground properties in a layered media. It can be used for loosening loads in rock or for gravity loading in soils. However, the linear analysis of this type will always provide conservative designs because the strength will be larger than that computed. In the next section, the difference between the linear and nonlinear analysis is discussed for the beam-spring model, and quantitative comparisons are made in Chapter 3.

Load is applied directly to the lining in the beam-spring model, causing it to oval or deform outward at the springlines and causing a resisting pressure from the medium that is proportional to the lining deformation; thus the first component of interaction described in Section 1.3.1, due to overall deformation, is satisfactorily modeled. Since the load is applied directly to the lining, the second component (change in shape of the active pressure due to local arching) is not taken into account; however, if the shape of the applied load can be estimated, it can be given the proper shape when it is applied. Neither of the interaction components three or four are automatically considered, but they can be included approximately if they are considered to have a significant effect; this would be done by applying horizontal pressure to the lining just as the vertical active pressure is applied to represent the medium pressure due to the Poisson effect and by modifying the vertical load to account for overall arching around the lining.

1.3.4 Nonlinear Analyses

When the loads on a lining become large enough to cause the concrete to crack and at higher loads cause the concrete compressive stress to become nonlinear or the reinforcement to yield, the lining becomes less stiff at these locations and the moment is reduced or does not continue to increase. The overall stiffness of the lining relative to the medium is reduced and the lining deflects at a greater rate than before these events occurred. The resulting additional deformation causes an increase in the passive pressure of the medium at the sides and causes the pressure around the lining to become more uniform; consequently, more of the additional load is resisted by a pure thrust mode in the lining.

When cracks form and there is limited tension at the section because the reinforcement yields or there is no tension in unreinforced linings, the resistance to thrust is similar to that in an unbolted, segmented lining or brick arch, except that the lining is likely to be thinner than these structures that are designed as pure compression arches and proportioned so that the thrust remains within the center portion of the section. Cracking implies that no stress occurs over part of the section and so the area in compression is reduced. As deformation of the lining increases the unstressed area increases and the area in compression decreases. During this process, the resisting moment does not increase very much with increasing load, but the thrust may increase significantly. A load is finally reached in which the increased thrust and decreasing compression area result in compression failure of the concrete; this failure is closely related to the lining deformation because it is related to the deformation of the failure section and thus the area of concrete in compression; the lining deformation is closely related to the stiffness of the medium around the lining.

The load at which failure finally occurs is larger than that which would be predicted by a linear analysis and the difference depends largely on the relative stiffness of the medium and lining, which can be indicated approximately by the flexibility ratio. The difference is larger at small values of flexibility ratio and decreases as the flexibility ratio increases. For large values of flexibility ratio that result for linings in rock, the linear analysis is probably adequate for design, but for linings in softer materials it may be more economical to take advantage of the nonlinear increment of strength.

There are several general purpose programs available that will perform nonlinear analyses of structural frames and can be used for the lining analysis. A program developed for this project is based on using a series of straight beam elements for the lining that use the nonlinear concrete and reinforcement stress-strain curves to describe the beam behavior and include the effect of geometric nonlinearity by continually updating the coordinates of the lining. This program allows the concrete stress-strain curve to have a descending branch after the peak strain, which is necessary to predict the peak lining load. Most other nonlinear programs do not allow the descending branch in describing the material properties.

This same lining representation is used with radial and tangential springs to represent the medium (the beam-spring model) or it is used with continuum elements to represent the medium (the beam-continuum model). The springs in the former model and the continuum elements in the latter remained linear. However, an interface or joint element can be used between the lining and medium in the beam-continuum model that has limited shear properties described by cohesion and angle of internal friction parameters. If the beams are given linear properties in this model and the interface elements are given full slip or no slip properties, it provides essentially the same results as the closed form solutions when the same loading is applied (Section 3.3 of Volume I). There are small differences resulting from the discretization of the lining and the medium that become smaller as the beam and continuum elements are reduced in size.

The reason for using the beam-continuum model of this type is that it offers versatility in working some types of problems that cannot be handled by other solution methods. The beam-continuum approach allows consideration of depth of cover, noncircular lining shapes, partial slip as well as full and no slip between the lining medium, a nonuniform or stratified medium, nonlinear behavior of the lining and nonlinear interface stresses between the medium and lining. Also, the properties of the lining can vary around its circumference to account for cracking, joints, or a variation in design which the closed form solution cannot handle. It also has all the advantages of the beam-spring model but in addition, the excavation and overpressure types of loading that result from the combined deformation of the lining and medium can be handled. Therefore, there are certain worthwhile advantages to a beam-continuum solution that may be desirable if the design problem warrants their use. This model includes all four components of interaction described above. It will also provide the additional strength of the lining that results from nonlinear lining behavior and the consequent redistribution of moment. It will not allow a void to occur between the lining and medium before contact between the two occurs, nor will it allow the medium to have nonlinear behavior unless nonlinear continuum elements are incorporated.

CHAPTER 2

EVALUATION OF EXISTING DESIGN PRACTICE

2.1 PURPOSE AND METHODOLOGY

Sixteen design firms that are engaged in tunnel design of various types were interviewed to survey the existing procedures used for design of underground concrete structures. This survey served the purpose of: (1) providing insight into the existing design methods, (2) showing where uncertainty exists in the design procedures and therefore where research would be most fruitful, and (3) indicating how new developments can be formulated to fit into the existing design framework.

Initially, the literature was surveyed in an effort to determine the procedures used by designers, but it became clear that most of the reported information concerns the analysis techniques used and many of the assumptions and simplifications and their implications on the design are not discussed. Consequently, it was decided to conduct a survey by visiting various design firms around the United States in the fall of 1980. The information described in this report is derived from discussions with the tunnel designers. Since the aim of this research effort is to propose design guidelines for transportation tunnels, most of the designers interviewed were engaged in this type of work but a few designers of water conveyance, hydroelectric, and sewer tunnels were also consulted to broaden the scope of the study and to show the influence of the function of the tunnel on the design procedures. Furthermore, because of the intimate relationship between the design and construction processes, the contractor's point of view was taken into account by visiting representatives of two contracting organizations that are currently engaged in the construction of tunnels.

There is a great diversity of design procedures used, even more than expected at the outset. It may be said that no two firms use the same procedures for design, though there may be some similarities in part, such as determination of the loads or the type of analysis used. For this reason, a summary is rather difficult. Also, there is always some confusion during the discussion concerning what the firm actually does and what the representative feels they should do if they had the time or money or were working under ideal conditions. There is sometimes a lack of agreement among individuals in the same firm. In some cases, the views of the representative have been changed by the last job, and the procedure used on the next job would be different. An effort was made, however, to determine an overall design philosophy used by the firm.

The firms and individuals visited were selected because of their involvement with recent tunnels and are listed alphabetically in Table 2.1. It was not possible to contact all tunnel designers in the United States, but it is believed that those interviewed constitute a representative group. An effort has been made to report as accurately as possible the design approaches that were discussed in each case strictly to compare design methods and not to criticize or judge any procedure.

A brief summary of the findings from the interviews is presented in the remainder of this Chapter, and more detailed descriptions may be found in Appendix A of Volume I. For convenience in the discussion, the design approaches are divided into loading, analysis, and criteria for strength and serviceability. Within each of these categories, the discussion is further divided into running tunnels in rock with a diameter of 30 ft (9 m) or less, large openings in rock with spans of 40 to 60 ft (12 to 18 m) and running tunnels in soft ground. The main reason for the distinction between opening sizes in rock is the effect of opening size, method of excavation, and type of initial support on the loading of final linings.

TABLE 2.1 LIST OF PARTICIPANTS

Firms	Designers Interviewed	Recent Tunnels Discussed
1. Bechtel (San Francisco, CA)	Walter Ferris Art Arnold Chris Gardner	Sultan Tunnel, Washington State (Designer). Also acting as construction manager in various tunnel projects.
2. DeLeuw Cather (Washington, DC)	Dah Fwu Fine Kuldip Singh	Washington Metro (General Engineering Consultant)
Bechtel (Bethesda, MD)	Carl Bock	Washington Metro (Construction Consultant)
3. Denver Board of Water Commissioners (Denver, CO)	John Parsons Verne Hornback Jim Batt W. Colwell, Jr. (with DMJM Phillips-Reister-Haley, Inc.)	Foothill Tunnel, Roberts Tunnel, Arrow Tunnel, Colorado
4. D.M.J.M./K.E. (Baltimore, MD)	David Hammond Drupad Desai	Baltimore Region Rapid Transit System
5. Goldberg-Zoino Associates of NY (Buffalo, NY)	Richard Flanagan	Buffalo Light Rail Rapid Transit. (Geotechnical Consultant) Sewer Tunnel in Rochester
Hatch Associates (Buffalo, NY)	Wesley Terry	Buffalo Light Rail Rapid Transit. (Principal Engineering Consultant, Rock Tunnel Section)
6. H & A of New York (Rochester, NY)	Gary Brierley	Sewer Tunnels in Rochester, NY Portions of Boston Subway
7. Harza Engineering (Chicago, IL)	Jan Veltrop Arvids Zagars William Bristow William Shieh Jerry Hahn Ed Cikanek	Chicago's TARP Project Zoo Park, Cleveland Park and Van Ness Stations in Washington Metro and tunnels and shafts for various hydroelectric projects throughout the world.
8. Jacobs Associates (San Francisco, CA)	James Wilton	Primary support for San Francisco sewer tunnel, Washington Metro (section D-9)
9. Jenny Engineering (South Orange, NJ) (Milwaukee, WI)	Prakash Donde Lloyd Monroe	Milwaukee sewer tunnel (Soft Ground) Sewer in Rochester (Rock) Sewer in Bankong (Concrete Segments in Soft Ground) Redesign sewer tunnel in Detroit

TABLE 2.1 LIST OF PARTICIPANTS (Continued)

Firms	Designers Interviewed	Recent Tunnels Discussed
10. Leeds, Hill and Jewett, Inc. (San Francisco, CA)	John Bischoff	Eisenhower Tunnels, Colorado
11. New York City Transit Authority (New York City, NY)	Mel Oberter Abe Blumberg Richard Mitchell	New York City Subways
12. Parsons-Brinckerhoff (New York City, NY)	Tom Kuesel Dan Wallace Bill Daly Bill Hansmire	BART System, Lexington Mkt. section in Baltimore WMATA (F2A,C4), Atlanta Subway (N120)
13. Parsons-Brinckerhoff/ Tudor (Atlanta, GA)	Douglas J. Mansfield Zdenek Zachar Perry M. Lin	Atlanta subways
14. Tudor Engineering (San Francisco, CA)	Don Rose Heinz Mueller Richard Mayes	San Francisco cross town tunnel
15. U.S. Army Corps of Engineers; Missouri River Division (Omaha, NE)	Dan Hokens	Primarily water conveyance tunnels
16. Bureau of Reclamation (Denver, CO)	Paul Tilp Timothy Smirnoff Ken Schoeman	Hades and Rhodes tunnels, Utah Stillwater tunnel, Utah

2.2 LOADING

2.2.1 Tunnels in Rock

Most running tunnels in rock are supported temporarily until the excavation is completed and then the final lining is installed in the stable opening. Ten of the firms visited select the final lining on the basis of minimum thickness for convenient construction and do not make calculations based on ground loads. Some designers reason that the temporary support has stabilized the opening and the final concrete lining is only needed to maintain that stability, to provide leakage control or to fulfill other owner requirements. Others reason that the capacity of a lining with nominal thickness is more than adequate for most cases, even if loads were applied. It should be pointed out that some of the firms who make this assumption design sewer or water conveyance tunnels in which case the size may be smaller than transportation tunnels and the owner requirements less strict.

Two firms who primarily design transportation tunnels select typical rock wedges based on the site geology for loading determination, while three others use an index of rock quality such as the RQD (Rock Quality Designation), RSR (Rock Structure Rating) or Terzaghi's loads or some combination of these for the design. One uses the effective overburden and designs the lining to resist this load in thrust. In these cases the loads are used to design the final lining, and the initial supports are to a large extent left to the discretion of the contractor.

All six of the design firms who select rock loads also consider full or partial water pressure acting on the lining in conjunction with the rock loads. In addition, six of the firms who assume that a minimum lining is adequate, do check for water pressure loading while the rest assume that it is adequate for external water pressure as well as rock loads. One firm drains the tunnel whenever possible and does not consider water pressure at all in those cases. A separate analysis for internal pressure is always performed if it occurs. It is not clear to what extent the other firms consider drainage, but those who mentioned partial water pressure are probably considering that some drainage occurs. Noncircular tunnels with long straight walls are more likely to be drained to avoid stress concentration at the corners. In view of the recent problems with clogging of some of the drains in Washington, D.C. and Baltimore subways, the whole water pressure loading philosophy is uncertain at this time. Dynamic and live loads are rarely (if ever) considered.

Ten of the firms had designed large openings in rock and obviously their philosophy varied with the type of rock and function of the opening. Two of the firms who have designed power plant chambers could select the site and orientation of the opening to be most favorable and so they felt that the need for a final cast-in-place concrete lining is questionable with proper initial support. This could also be argued for transportation tunnels in competent rock (i.e., Atlanta subway), though the function may introduce other considerations such as aesthetics, water seepage, ownership and legal requirements. Five of the firms who have recently designed subway stations in less competent rock (i.e., Washington, D.C. subway) based loads on local geology and the selection of rock wedges or full overburden. Two firms have based their design loads on the RQD concept or some modification of it, while two others have used a minimum loading on the final concrete lining that they base on the excavation and initial support sequence. They advocate controlling these activities carefully so that the initial support actually performs the primary support function.

Of the six designers who select rock loads based on rock wedges or RQD, four of them consider full or partial water pressure with the rock loads. Four designers prefer to drain the external water pressure, while the remaining two would either design for water pressure or drain the water depending on the site conditions and function of the opening.

2.2.2 Tunnels in Soft Ground

Six design firms provided information on loadings for tunnels in soft ground of which three designed transportation tunnels and three sewer tunnels primarily. Of the transportation tunnel group who designed cast-in-place final concrete linings, one used a full overburden vertical load accompanied by a horizontal pressure of 0.875 times the vertical pressure for a working stress design and then the lining ultimate strength was checked with a reduced lateral pressure. Another used 1.5 to 2.0 diameters of soil weight above the crown for the initial support and 60 percent of this value for the final lining and used an analysis that represents the soil with springs, so the lateral pressure depends on the soil stiffness. The third in this group prefers to carefully control the excavation and initial supports and then consider the minimum constructable final lining to be adequate.

Of the group who designs cast-in-place sewer tunnel linings, the first uses full overburden reduced by soil shear resistance if the final lining is installed within a few days of the initial support, and full overburden if it is installed later. Lateral pressure near the at rest earth pressure is applied. The next designer uses full overburden up to 80 ft (24.4 m) depth and some portion of full overburden above this value with a lateral pressure between the passive and at rest earth pressure values. Alternatively, he may determine moments by imposing a diameter change on the lining that depends on the tunnel size. The last designer in this group considers the minimum constructable lining adequate if care is exercised in providing flexible initial support.

Five design firms provided information on segmented precast lining design and all advocated first proportioning the segments for handling and jacking loads and then checking for ground loads not greatly different from those used for cast-in-place linings or checking for thrust due to full overburden and moments due to a specified diameter change. They emphasized, however, that care must be exercised in the installation to obtain good contact with the ground through competent backpacking and grouting without appreciable ovaling.

Most of the designers include the effects of external water pressure and internal water pressure as well, when present.

2.3 ANALYSIS

2.3.1 Tunnels in Rock

Nine firms consider the minimum constructable cast-in-place lining in running tunnels adequate and do not perform an analysis, and of the seven remaining, two of them use a frame analysis in which they represent the lining by a series of beam elements and the medium by springs, and one uses a similar frame analysis for the lining and finite continuum elements for the medium (these are larger than normal running tunnels, however). The remaining designers use either closed form solutions for a cylinder embedded in an elastic medium, design charts and experience or a design for thrust based on full overburden and moment based on an eccentricity that is some portion of the radius of the tunnel.

Eight of the firms described their analysis for large openings in rock and emphasized that the geology at the site and function of the opening influenced the extent of analysis required and even whether an analysis is necessary. Five of the firms who designed large openings used linear beam elements for the lining and four of them used radial and tangential springs for the medium, while the other used plane strain two dimensional continuum elements in modeling the rock. Another design group uses the linear beam element for the lining but does not allow the rock to deform at the sides of the opening. When springs were used for the medium, the value of tangential spring stiffness varied from zero to 50 percent of the radial stiffness. The remaining firms perform simple analyses to check the lining strength and depend primarily on experience and control of the excavation and initial support sequence to assure stability of the opening with minimum final support; in some cases the ground and excavation and support sequence were modeled with a finite element program to assure stability during excavation and help determine amount of initial support needed.

2.3.2 Tunnels in Soft Ground

Five firms provided information on their design approach to linings in soft ground. Three firms use linear beam elements to represent the lining, radial springs for the medium that are detached if they are in tension, and do not include tangential springs. Two of this group also include a lateral pressure to represent soil stresses acting in addition to that from deformation of the lining and when this is done, full overburden or nearly full overburden is used as the vertical load. Another design group does not include the additional lateral pressure but the vertical load is limited to 1.5 to 2.0 diameters of soil, the load depending on the type of soil present. Full or partial overburden soil pressure applied uniformly around the lining to obtain the thrust and a predetermined diameter change to calculate the corresponding moment in the lining is used by another design organization. A variation of this method is also used in which the thrust is obtained using the same approach but the moment is computed by applying this thrust with a predetermined eccentricity that is some portion of the lining radius. Another firm uses the closed form solution without shear stress between the medium and lining presented by Peck, Hendron and Mohraz (1972).

Those designers who use the beam-spring model for cast-in-place linings use the same model for segmented linings and reduce the stiffness of the joints to zero or a low value. Both firms who design for a uniform soil pressure and a moment from a fixed diameter change or eccentricity also use the same general approach for segmented linings. One firm uses an equivalent cracked section stiffness (EI) to represent the joints in the lining. In all cases, the segments are first designed for jacking forces, handling stresses and perhaps grouting pressure, and these considerations usually govern the design. One of the firms also checks stresses in the skin of the lining between ribs as an edge supported plate subjected to uniform pressure; they find that this check may frequently produce the largest stresses.

2.4 DESIGN CRITERIA

The strength criteria are not independent of the analysis used or loads selected, but for purposes of comparison they will be divided into the working stress method, in which actual expected loads are selected and stresses in the lining are limited to some portion of the material strength, and the ultimate strength method in which the working loads are multiplied by load factors and the ultimate strength of the lining is considered.

Thirteen firms described their strength criteria and of these seven used working stress methods and four used ultimate strength methods while two used both. One of these latter two firms uses either approach depending on the owner's requirements; the other proportions the lining using working stress methods and then checks it for ultimate strength with reduced lateral pressure for running tunnels and with increased water pressure for stations. Working stress values vary from $(0.25)\ f_c$ to $(0.45)\ f_c$ for the concrete and $(0.5)\ f_y$ to $(0.66)\ f_y$ for the reinforcement. Load factors used in the ultimate strength design method vary from 1.2 to 3.0 depending on the load type.

Serviceability criteria in terms of leakage and crack control were emphasized by the designers involved in transportation and sewage tunnels, while it is of little concern for designers of water conveyance tunnels. Most designers agree that for mass transit projects, leakage specified by an allowable leakage criterion can be tolerated in running tunnels, but it should be near zero for stations without an interior lining. Cracking in linings is acceptable as long as the leakage criteria are not exceeded. Nearly all designers specify the use of a grouting program to seal cracks that do occur but other measures to control cracking and leakage vary. Some advocate use of drains, whereas others use reinforcement to limit cracking and the resulting leakage.

There is wide disagreement among designers when it comes to the use of reinforcement in concrete tunnel linings. All the designers of transportation tunnels except two advocate the use of reinforcement in the circumferential and longitudinal directions for both soil and rock tunnels, but some use two layers and others only one. One designer typically uses one layer on the tension face for linings in rock and two layers for linings in soil. Most of the sewer tunnel designers use one layer of reinforcement. The water conveyance tunnel designers use no reinforcement at all, except in unusual ground conditions or when the tunnel is pressurized. Some designers feel that some reinforcement is necessary to prevent chunks of deteriorated concrete from falling out, especially for larger diameter tunnels, and to assure the long-term integrity of the lining.

Construction related criteria vary, depending on site conditions, construction practices in the area, owner requirements and so on. Four of the transportation tunnel designers specified lengths of casts that varied from 20 to 50 ft (6 to 15 m) with vertical construction joints to control shrinkage cracking. Several indicated that they thought a sloping joint at the angle of repose of the concrete would be more economical and would not cause any more leakage or other problems. The contractors were convinced that sloping joints, if properly made would cause less leakage problems. Some designers specify an interval, in terms of time or distance, between excavation and placement of a cast-in-place lining to assure that the opening has stabilized with the initial supports to avoid cracking of the final lining.

2.5 AREAS OF UNCERTAINTY IN DESIGN

Reinforcement: The greatest uncertainty among designers appears to concern the use of reinforcement in the final concrete lining. Many designers advocate the inclusion of reinforcement in the final lining without a valid justification. The argument that it contributes to the long-term integrity of the final lining used by some designers is challenged by others saying that reinforcing bars corrode with time, increase in volume and thus cause spalling in the concrete, in essence causing deterioration rather than preventing it. In many cases it is not clear whether reinforcement is needed to resist ground loads, to control shrinkage cracking or both. Even in the case when it is decided that reinforcement is needed, there is uncertainty whether to use a double or a single layer, closer to the inside face or in the center of the section. There is general agreement, however, that costs could be reduced significantly if reinforcement could be left out, except in those cases where it is definitely needed.

External Water Pressures: Handling of external water pressure on linings is also uncertain in terms of when to design for it and when to drain the area around the lining to reduce the pressure. There is a tendency to try to drain large openings in rock and tunnels with sharp angles or long straight walls. In these cases, there is the possibility of high external hydrostatic pressures creating stress concentrations or high tensile stresses.

However the decision to drain does not necessarily resolve the issue. Recent problems with clogged drains behind some tunnel linings in Washington, D.C. and Baltimore subways indicate that the question is not whether the tunnel ought to be drained but whether it can be drained over the entire life of the tunnel. Therefore, careful analysis of the chemistry of the ground water and provisions for maintaining the drain openings were mentioned as important design considerations.

Loading for Linings in Soft Ground: The diversity of approaches used in the design of the final concrete lining for tunnels in soft ground indicate a lack of agreement among engineers. Most designers agree on the concept of placing the initial flexible support and allowing the opening to deform until it is stable before the final cast-in-place concrete lining is placed. Opinions vary on the loading that then occurs on the final lining. The magnitude of the ground loads applied over the crown are usually related to the nature and magnitude of the assumed side reactions. When full overburden is selected as the vertical load, a fairly large portion of the vertical pressure is applied horizontally; when the vertical load is limited to 1.5 to 2 diameters of soil weight, lateral pressure is assumed to occur only due to deformation of the lining.

<u>Analysis of Precast Segmented Concrete Linings</u>: Because of the lack of experience with precast segmented concrete linings in this country, there is uncertainty concerning the approach that should be used for their design. It is generally believed that designs based on handling, erection and jacking forces will also be adequate for ground loads. It was pointed out, however, that the joints are sometimes significantly weaker than the cross-section of the segment, depending on the joint details, and in some cases may become a critical section under ground loads.

<u>Cracking and Leakage</u>: Cracking of the final concrete lining, per se, is not considered to be a problem unless it is related to leakage. Some designers consider the tunnel lining satisfactory even if cracked, as long as it does not leak. In sections of transportation tunnels in particular, where people are present, leakage control is considered essential. Unfortunately, the designer is not in a position to guarantee a dry tunnel during the design stage, unless a well proven drainage system, or a shield interior lining is used. Certain designers attempt to specify "low" stresses in the longitudinal reinforcement to control load-related cracking and minimum amounts of longitudinal reinforcement to control shrinkage-related cracking. A lot of designers feel, however, that instituting a post-construction crack treatment program is the most effective way to control leakage.

<u>Degree of Conservatism In the Design of Larger Openings In Rock</u>: As the size of a tunnel opening in rock increases, so does the degree of uncertainty with regard to the loads that will reach the final lining and the degree of conservatism in the overall design. The possibility of large rock blocks sliding along discontinuities in the rock mass

(joints, shear zones, etc.) and applying relatively concentrated or eccentric loads does exist. Furthermore, the excavation and initial support sequence become extremely important in determining the magnitude of rock loads that are likely to reach the final lining. Also the consequences of problems during construction and after the facility is in use have far reaching financial implications. For these reasons, the designer generally is very conservative in his approach.

Effect of the Initial Support System on the Final Concrete Lining: Different philosophies exist as to whether to account for the initial support system or not and if it is considered, how much strength it provides to the lining system or how much it reduces the load on the final concrete lining. If the initial support system is considered, then the loads on the final concrete lining are usually reduced by an amount which depends on the type of the initial support used.

The research effort was directed toward providing answers to these areas of uncertainty in the design of the final concrete linings. Not all questions have been answered, however, since some of the existing uncertainties will be resolved only through long-term field measurements, and their correlation with analytical and experimental results.

CHAPTER 3

SUMMARY OF RESEARCH AND DESIGN IMPLICATIONS

The research consisted of a series of model tests on arch and circular linings and development of a finite element analysis that was used to study effects of parameter variation on arch linings in rock and circular linings in rock and in soft ground. Volume I contains detailed descriptions of these studies as well as background information and implications on design. A summary of the results and their design implications is presented in this chapter to provide the reasoning behind many of the design recommendations made in Chapter 4.

3.1 MODEL TESTS

3.1.1 Arch Linings In Hard Medium

Test Description: Two types of models were tested to evaluate certain behavioral characteristics of arched linings in rock. In particular, the effects of loading shape, tangential shear between lining and medium, flexibility ratio and lining reinforcement on the overall behavior, and failure mechanisms of the lining were evaluated. It is reasonable to compare the effects of each variable among these tests when all other variables were the same; though the scaling of information to the full scale structure in the ground, discussed in Section 2.3 of Volume I, may be questioned, scaling can still give some idea of the effects of the same variable in a full scale system. The results and conclusions from these tests will be summarized in this section.

Ten arches, 6 ft (1.8 m) in diameter and varying thickness, surrounded by a concrete medium to represent rock, were tested. They were loaded by applying forces with hydraulic rams directly to the upper 60 deg segment centered on the crown to simulate loosening loads. The test arrangement and loading system are shown in Figure 3.1a. Two

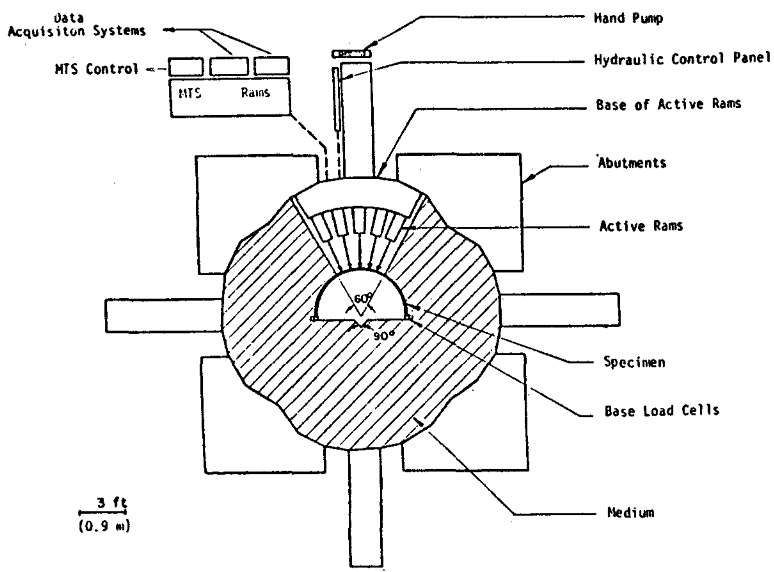


FIGURE 3.1a PLAN VIEW OF TEST SET UP

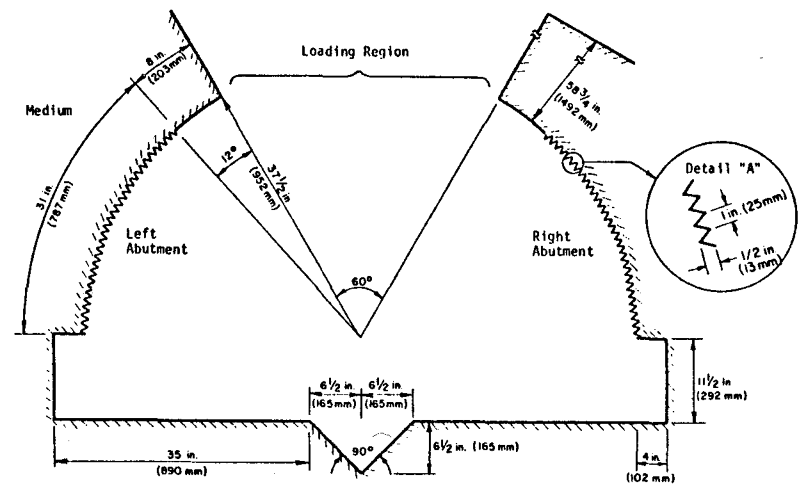


FIGURE 3.1b VIEW OF THE SERRATIONS

FIGURE 3.1 TEST SET UP FOR ARCHES

medium stiffnesses were used, first by using only the concrete and second by replacing some of the concrete by neoprene pads; the actual in-place effective medium stiffness was determined with plate load tests. Shear deformations between medium and lining were prevented in some tests by casting serrations in the medium as shown in Figure 3.1b and in one other shear stress was reduced by providing a smooth lubricated surface. The hydraulic ram forces could be varied to maintain the load shapes shown in Figure 3.2.

Table 3.1 summarizes the variables and Table 3.2 provides a summary of the most significant test results. In these comparisons strength is normalized as the ultimate thrust in the lining at failure T_u divided by the ultimate axial thrust or that which would occur if there was no moment, T_o. The reason for this normalization is that the strength ratios can be compared among tests when the concrete strength is different or the reinforcement is present or not, in order to determine the effects of other variables. Reduction of this ratio from the value of one is due to the presence of moment at the failure section, and can be viewed as a relative strength without regard to the actual load and how it occurred. The strength ratio is shown on Figure 3.3 as a function of the flexibility ratio F for all the tests. The flexibility ratio has been derived as a measure of equivalent stiffness of the medium to that of the lining in flexure for a circular lining (Peck, Hendron and Mohraz, 1972) and the formula is given in Section 4.3.4. This derivation is not applicable to arches directly, but it still provides a convenient measure of this relative stiffness and contains the appropriate variables; therefore, it is used here as the variable describing the relative stiffness of the lining-medium system.

Effect of Load Shapes: The load shape that gave the lowest T_u/T_o ratio at F = 1200 and interlocking between the lining and medium was the symmetrical triangle with load concentrated at the center. The largest T_u/T_o resulted from the uniform load. The range was from 0.60 to 0.69. However, arching in the field is likely to limit the magnitude of nonuniform loadings more than those that are uniform, making the uniform load the one that gives the lowest safety factor

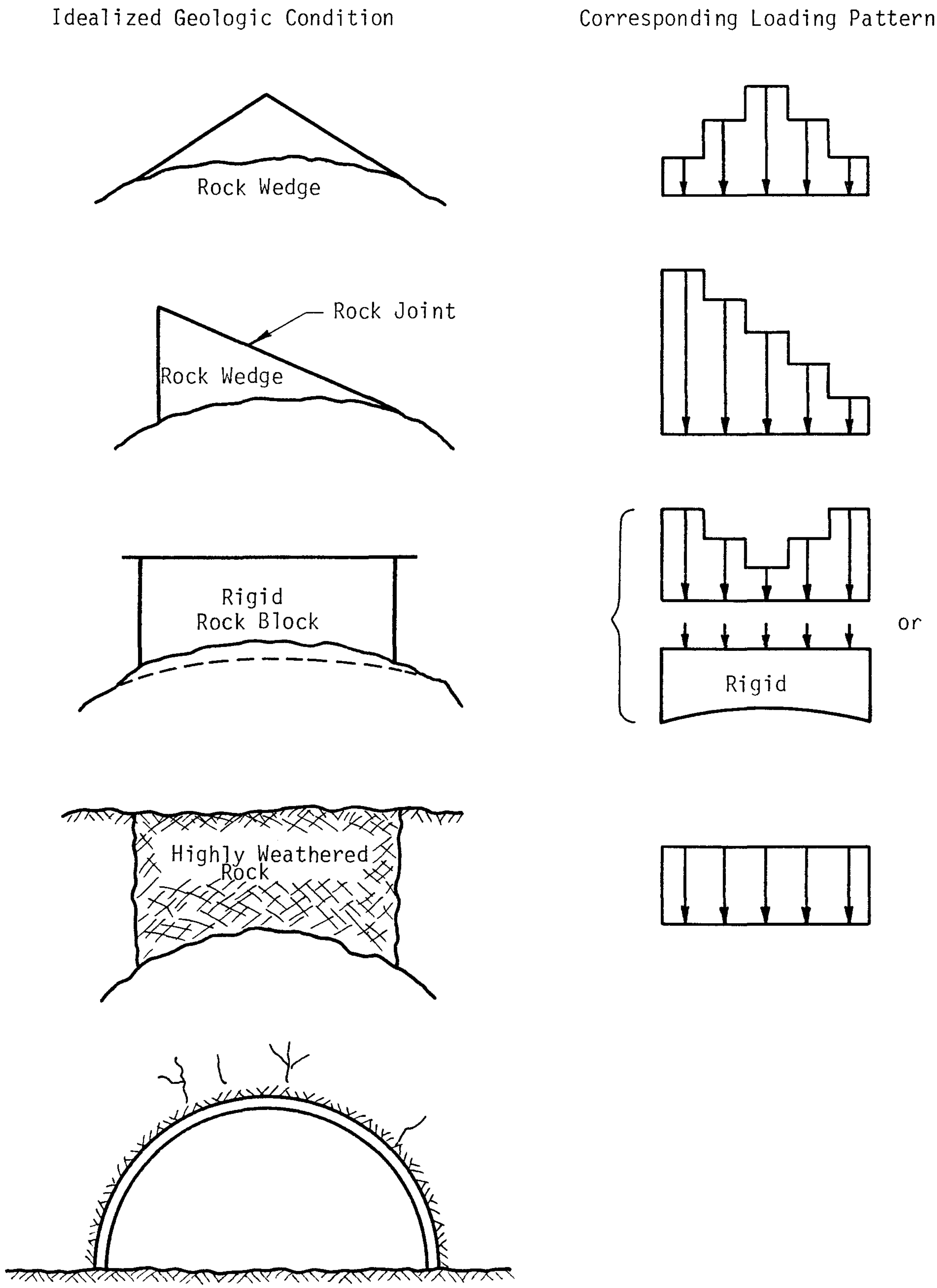


FIGURE 3.2 LOAD SHAPES USED IN ARCH TESTS

TABLE 3.1 SUMMARY OF ARCH TESTS IN HARD MEDIUM

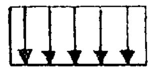
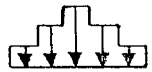
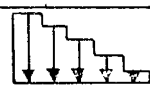
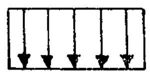
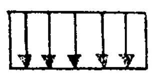
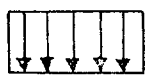
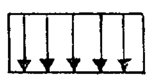
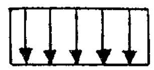
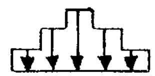
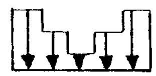
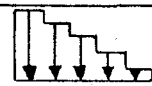
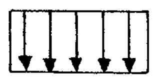
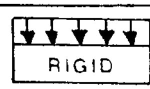
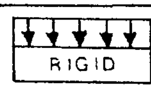
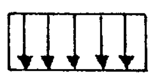
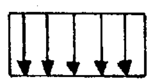
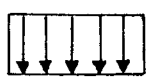
Designation	Loading Shape	Thickness in (mm)	Rock Modulus psi (MPa)	Tang. Shear	Reinforced
ARCH-1		3(76)	1.68×10^6 (11575)	YES	NO
ARCH-2		3(76)	1.68×10^6 (11575)	YES	NO
ARCH-3		3(76)	1.68×10^6 (11575)	YES	NO
ARCH-4		3(76)	1.68×10^6 (11575)	YES	NO
ARCH-5		3(76)	1.68×10^6 (11575)	NO	NO
ARCH-6		3(76)	1.68×10^6 (11575)	YES	NO
ARCH-7		3(76)	0.17×10^6 (1170)	YES	NO
ARCH-8		3(76)	0.17×10^6 (1170)	YES	NO
ARCH-9		1(25)	0.17×10^6 (1170)	YES	NO
ARCH-10		1(25)	0.17×10^6 (1170)	YES	YES

TABLE 3.2 SUMMARY OF ARCH TEST RESULTS

Test Designation	Loading Shape	Flexib. Ratio	Tang. Shear	Reinf.	Thrust Ratio T_u/T_o	$\Delta R/R\%$	Cracking Load % of Ultimate
ARCH-1					0.69	0.53	NO CRACKS
ARCH-2			YES		0.60	0.61	50
ARCH-3		1200			0.67	0.41	NO CRACKS
ARCH-4				NO	0.64	0.52	NO CRACKS
ARCH-5			NO		0.52	1.09	80
ARCH-6					0.64	0.30	NO CRACKS
ARCH-7		120			0.37	1.50	40
ARCH-8			YES		0.42	1.03	60
ARCH-9		3650			0.62	1.03	85
ARCH-10				YES	0.65	1.60	80

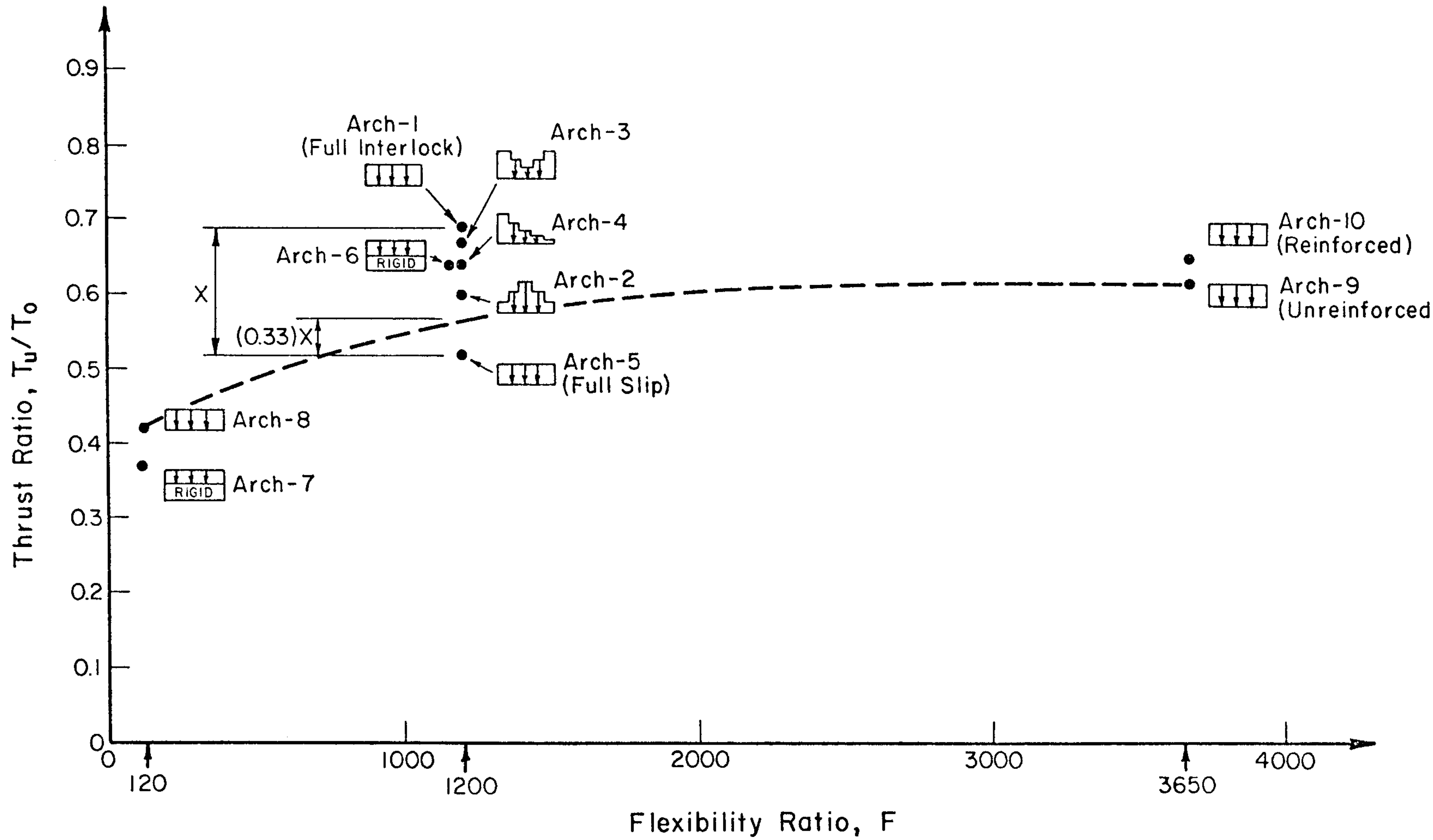


FIGURE 3.3 EFFECTS OF LOADING SHAPE AND FLEXIBILITY RATIO ON THE THRUST RATIO

against failure. At F = 120 loads were applied uniformly and from a rigid block, and the T_u/T_o values were 0.42 and 0.37. Failure of the specimen with triangular loading that peaked at the crown was ductile and resulted from flexure at the crown. Comparable tests with other shapes of loading failed more suddenly and at or near the edges of the loaded area. Load shape also had an effect on the deformability of the lining with the rigid block loading (Arch-6) exhibiting the smallest ductility as shown in Table 3.2.

Effect of Tangential Shear Between Lining and Medium: The effect of tangential shear is shown by comparing Arch-1 with full interlock and Arch-5 with no interlock, and both with the same F of 1200. Removal of interlock reduced T_u/T_o from 0.69 to 0.52. Removal of tangential shear leads to larger deflections, which allows the crown region to flatten more and creates a larger moment; it also increases greatly the force reaching the base of the arch. With interlocking the base force was from 5 to 30 percent of the total load, but without interlock this base force was slightly over 100 percent, a sizable increase as far as the design of the base footings is concerned.

Effect of Flexibility Ratio: The rock modulus and lining stiffness are incorporated in the flexibility ratio which has a marked effect on T_u/T_o as shown in Figure 3.3. There is a problem with the comparison, because at F = 1200 the full interlock and no interlock cases were tested, while at F = 120 and 3650 neoprene was used between the lining and medium which would be equivalent to a partial interlock between these extremes. By trial and error, in trying to match the analysis with experimental results, it was determined that these latter cases with the neoprene corresponded to a tangential shear stiffness that was about one-third of the radial stiffness, so the curve shown in Figure 3.3 is drawn 1/3 of the way between Arch-5 (no interlock) and Arch-1 (full interlock) at F = 1200. The resulting curve is then for the same tangential shear condition and uniform loading. Though exceptions can be taken with the value used, the curve appears reasonable, and for this loading case T_u/T_o varied from 0.42 at F = 120 to 0.62 at F = 3650. Also, the curve is rather flat beyond F = 2000, indicating that there is an upper limit for T_u/T_o, which is about 0.62 for the uniform load case in good quality rock. This conclusion is based on the assumption made at F = 1200 to draw the curve, but any assumption made will result in a fairly flat curve. It may reasonably be expected that there is a similar limit for other load shapes or other tangential shear conditions. A minimum amount of moment should be expected in the lining no matter how high the value of rock modulus. This moment causes the thrust ratio to drop from 1.0 (no moment) to about 0.62 for these test conditions.

Cracking: Cracking characteristics of the test specimens are summarized in Table 3.2 and Figure 3.4. Flexural tension cracks did not occur in four of the tests, and in all tests that had cracking except Arch-7, it first appeared at or above 50 percent of the ultimate load. In these cases if there were a safety factor against failure of at least 2.0, then cracking would not occur at service load. In reality cracks would be less likely to occur due to flexure in the field than shown by these model tests, because creep strains in the tests were small. When the compression zone of the concrete section creeps with time as the load is applied slowly, this zone shortens and the tension stress present on the other side of the section is relieved. Also, if the tension stress is applied over a long period, the concrete can creep in tension and a larger strain is required to cause cracking than would have occurred if the tension is applied more quickly.

There is a definite increase in the tendency toward cracking as the flexibility ratio becomes smaller as shown in Figure 3.4. This results from larger deformation and larger moment, which is consistent with the effect of flexibility ratio discussed above. In Arch-7 where cracking occurred at 40 percent of the ultimate load, the smallest flexibility ratio was combined with the rigid block loading that provided the greatest load concentration. In most of the tests in which cracking occurred, the width of the crack remained small during a considerable part of the remaining loading, and started to open significantly only near failure.

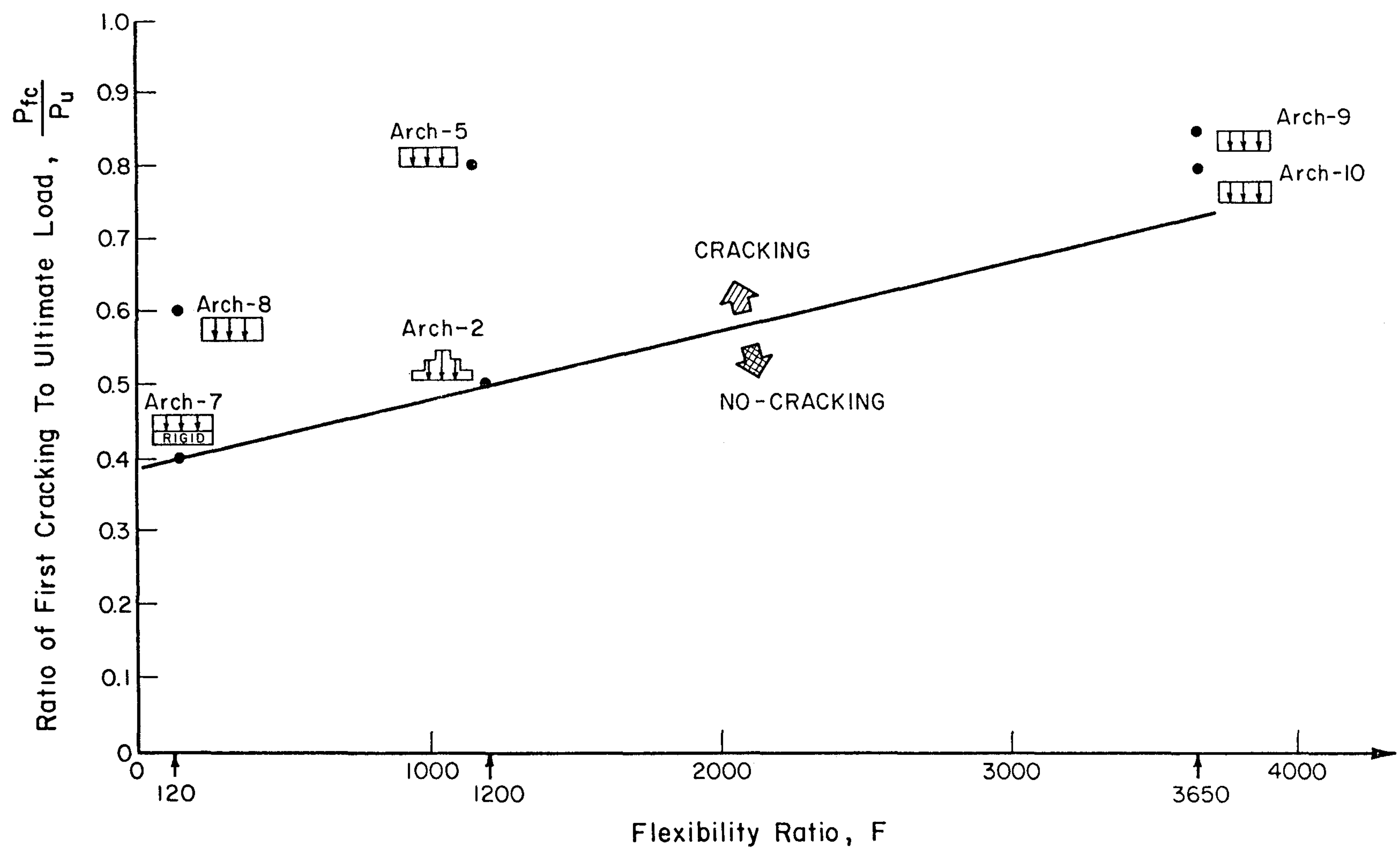


FIGURE 3.4 FIRST CRACKING AS A FUNCTION OF FLEXIBILITY RATIO

Effect of Reinforcement: Though reinforcement had little influence on the load capacity of the lining, it had a considerable influence on its overall behavior. The effects of reinforcement are assessed by comparing Arch-10 (1.0 percent reinforcement) with the companion test Arch-9 (unreinforced) and the rest of the arches. The normalized load to Arch-1 for Arch-9 was 44 kips (196 kN), while that for Arch-10 was 45 kips (200 kN). If all the bars reached their yield stress they could resist a thrust of 2.5 kips (11 kN) so it is reasonable that this thrust would result in a small increase in load. By comparing the general appearance of the cracks for the reinforced specimen with those for the comparable unreinforced ones, it appears that the reinforcement serves to distribute the cracks and by so doing keep them finer. When no reinforcement was present, only one or two cracks appeared in the high moment region near the crown or near the edge of the loaded zone, and when there were more than one, generally only one of them opened significantly while the others remained small. However, in Arch-10 with reinforcement, four cracks formed that were approximately evenly spaced and each of them opened at about the same rate. The cracks at ultimate load were four times as wide in the unreinforced specimen (Arch-9) than in the reinforced one (Arch-10). Also near failure reinforcement held the failed region together for a little more deflection at nearly constant load.

3.1.2 Circles In Soft Medium

Test Description: The purpose of the circle tests was to investigate the effects of reinforcement, medium stiffness and joints between segments on the overall structural behavior of linings in a medium with comparable deformability to soil. Five circular concrete linings, three monolithic and two segmented, 44 in. (1ltwo0 mm) in diameter, 1.0 in. (25 mm) thick and 12 in. (305 mm) long were tested. Loads were applied through the four center rams 3, 4, 5, 6 as shown in Figure 3.5. The remaining four rams 1, 2, 7, 8 provided passive support at the loading surface. A detailed description of the tests is given in Chapter 4 of Volume I and in Sgouros (1982). Since the primary interest was in behavior of the lining, the ground was

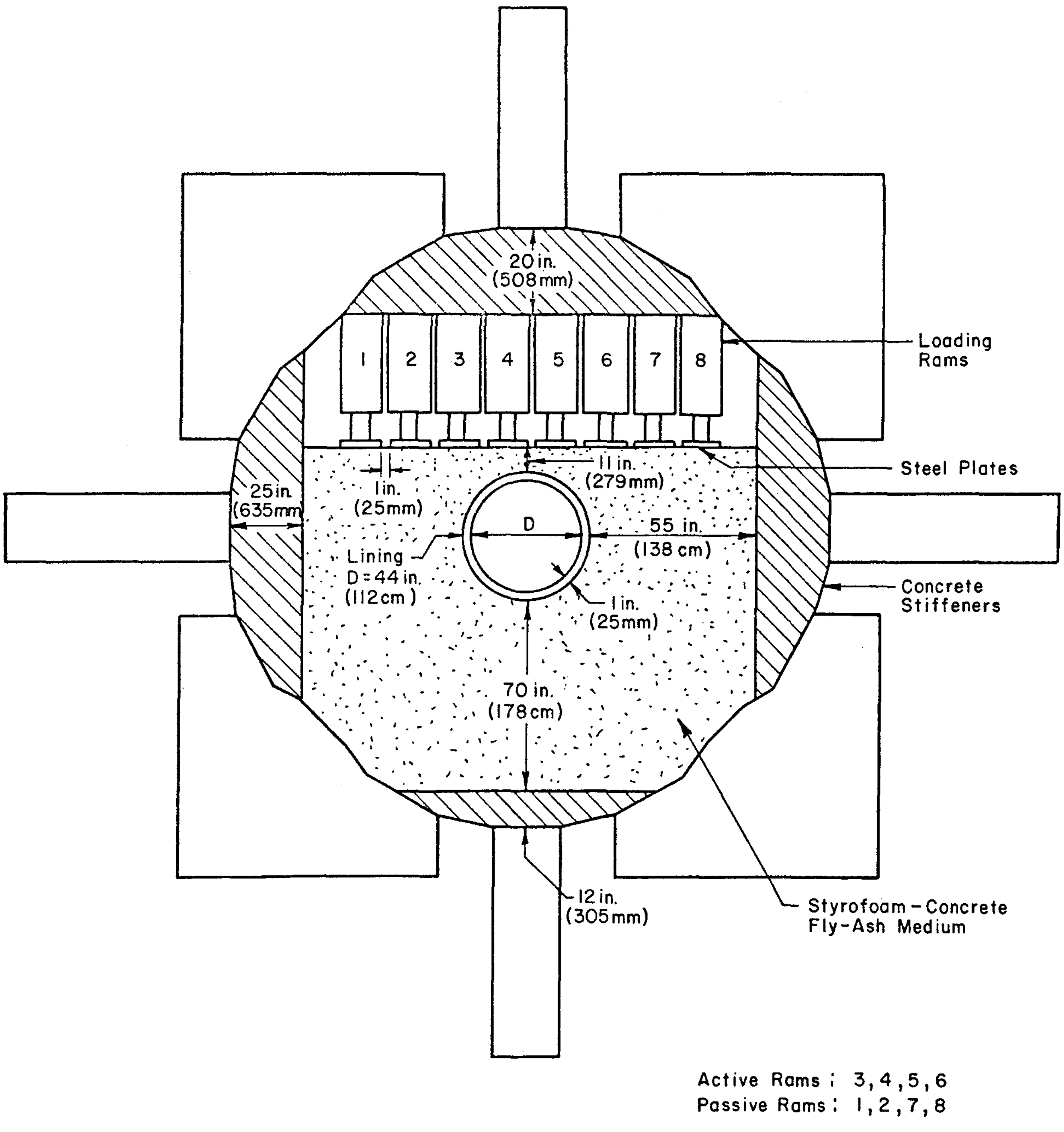


FIGURE 3.5 TEST SET UP FOR CIRCULAR LININGS

represented by a cement-fly-ash-styrofoam bead mix with the requirement that the deformation should be as large as a soil under the same load. An additional requirement was that the mix should have enough strength to transmit the loads from the loading rams to the lining. Because of this additional requirement the relative initial stiffness between the medium and the lining (expressed by the flexibility ratio F) remained high, as shown in Table 3.3, ranging from 170 for Circle-1 to 310 for Circle-4. However, because of the reduction in stiffness of the cement-fly-ash-styrofoam medium with load, it was estimated that the flexibility ratio near ultimate was as low as 10 percent of the initial values.

The circle diameter of 44 in. (1120 mm) as compared to a typical diameter of a subway tunnel of 20 ft (6 m) corresponds to a model scale of about 1:5. Applicability of model test results to full scale tunnels is discussed in detail in Section 2.3 of Volume I. In view of the limitations imposed by problems with producing and testing an exact scaled model, extrapolation of the test results to full scale tunnels should be done with caution. However, since all the tests were performed under the same conditions, the comparison between various models is reasonable.

Effects of Reinforcement: The effects of the amount of circumferential reinforcement on the load carrying capacity of the lining are examined by comparing Circle-2 with 1.0 percent reinforcement and Circle-3 with 0.6 percent reinforcement. As shown in Figure 3.6, the capacity of Circle-3 is higher even though the amount of reinforcement is lower, because of the slightly higher modulus of the medium: 40,000 psi (275.6 MPa) for Circle-3 and 35,000 psi (241.1 MPa) for Circle-2. Thus, the load carrying capacity of the lining is more sensitive to variation of the modulus of the medium than the amount of reinforcement present. Reinforcement does little to improve the load carrying capacity of the lining, because the reinforcement is most effective in tension, and for the flexibility ratio range considered, most of the lining section is in compression.

TABLE 3.3 SUMMARY OF MODEL CIRCULAR LINING TEST RESULTS

Type of Lining		Circle-1 Monolithic	Circle-2 Monolithic	Circle-3 Monolithic	Circle-4 Segmented	Circle-5 Segmented
Amount of Reinforcement (%)		0	1	0.6	1	1
Initial Equivalent Elastic Modulus of Medium psi (MPa)		25,000 (172.2)	35,000 (241.1)	40,000 (275.6)	45,000 (310)	35,000 (241.1)
Compressive Strength of Lining Concrete f'_c, psi (MPa)		2,760 (19)	2,560 (17.6)	2,240 (15.4)	2,720 (18.7)	2,280 (15.7)
Peak Load Kips, (KN)		48.4 (215)	73 (325)	90 (400)	77 (342)	70 (311)
Change in Diameter $\frac{\Delta D}{D}$ (%)	At 50% of Peak Load	0.36	0.43	0.25	0.41	0.36
	At 100% of Peak Load	1.27	1.20	1.30	1.20	1.86
First Flexural Cracking Load, % of Peak Load		42	36	32	91	93
Crown Crack Size at 90% of Peak Load, in. (mm)		0.01 (0.3)	0.001 (0.03)	0.004 (0.1)	--	--
Failure Section, Degrees from Crown		80-Left	70-Left	65-Right	60-Right	120-Left
Initial Flexibility Ratio, F		170	250	300	310[a]	260[a]
Estimated Flexibility Ratio, F at 90% of Peak Load		17	25	30	31	26
Thrust Ratio (T_u/T_o) at Failure		0.51[c]	0.56[c]	0.62[b]	0.56[b]	0.40[c]

[a]Monolithic flexibility, joints not taken into account.

[b]Measured.

[c]Estimated.

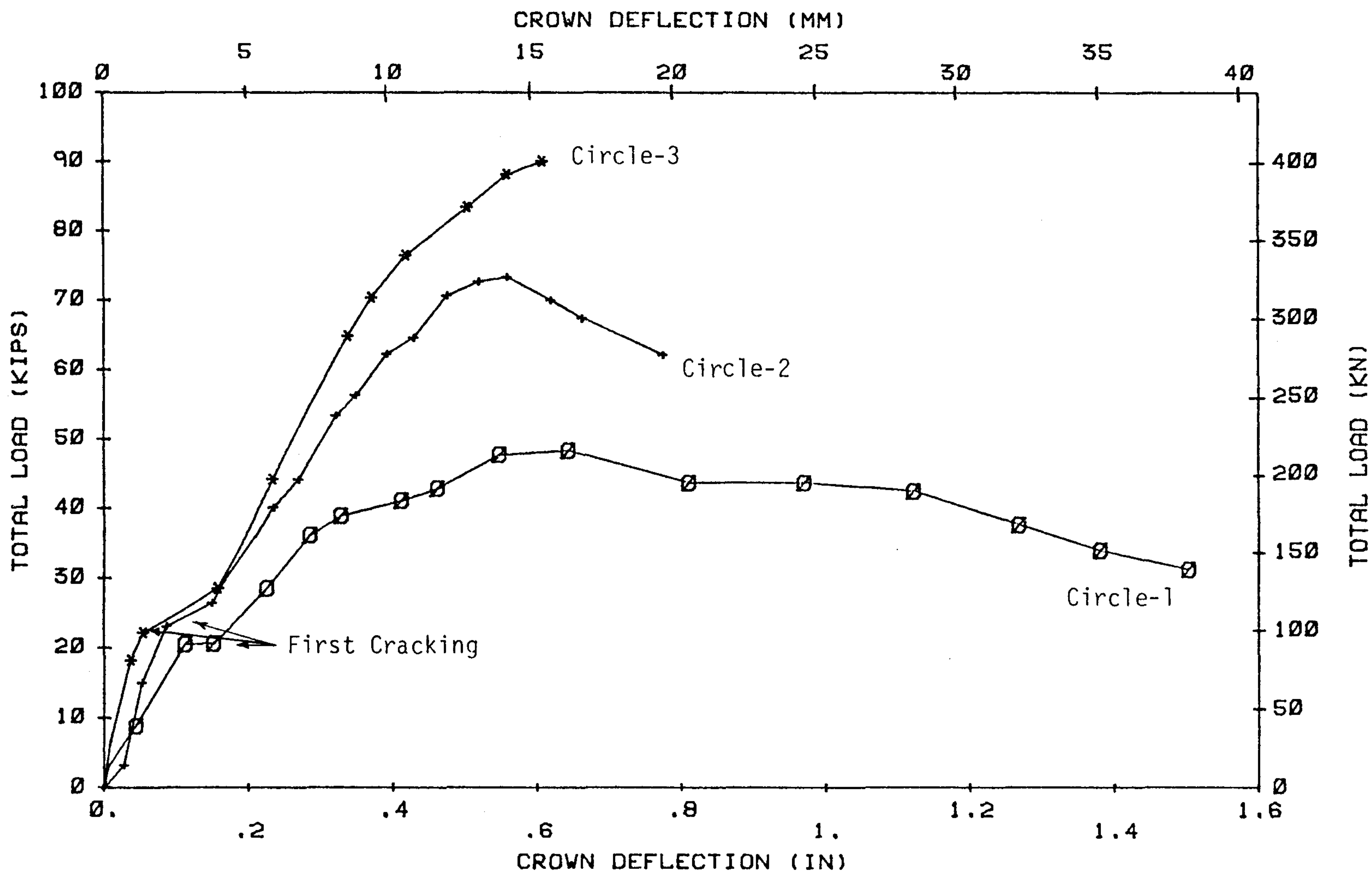


FIGURE 3.6 LOAD-DEFLECTION CURVES FOR THE MONOLITHIC LININGS

Flexure related cracks appeared at the crown and at the springline in all three monolithic lining tests. Overall additional cracking was more severe in Circle-1 (Figure 3.7) than in Circles-2 and Circle-3 consistent with its low modulus and lack of reinforcement. Comparing Circle-2 to Circle-3 in Figures 3.8 and 3.9, it is observed that less cracks are present in Circle-3 even though the reinforcement ratio is smaller, because of the larger medium modulus. The beneficial effects of reinforcement are observed in Figure 3.10, where the width of the crown cracks for the three monolithic linings is compared. Though initial crack width is less than 0.004 in. (0.1 mm) in all three linings, the crack opens with additional load in the unreinforced specimen but maintains the same width in the reinforced ones. At about 90 percent of the peak load the crack is three times wider in the unreinforced specimen.

The time of appearance of first cracks is affected very little by the amount of reinforcement, since it is a function of the cracking strain of the concrete. First cracks appeared at about 32 to 42 percent of the peak load in all three monolithic tests as shown in Figure 3.10. First cracks in the two segmented linings Circle-4 and Circle-5 appeared at 91 and 93 percent of the peak load. However, this was the result of the presence of the joints rather than of the reinforcement in the lining. The conclusions regarding reinforcement concern short term load-related cracking. The effects of reinforcement on shrinkage or temperature related cracking or the long-term behavior of the lining cannot be obtained from these relatively short term tests. If the loads were applied slowly and the concrete could creep during the load application, the cracks would occur at higher loads and would not open as much.

Effects of Medium Stiffness: The stiffness of the medium is the most important parameter in determining the load carrying capacity of the lining. The higher the modulus of the medium, the higher the capacity of the lining as shown in Table 3.3 for Circles-1, 2 and 3. A stiffer medium decreases the lining deformation and thus the moments, so the thrust ratio T_u/T_o increases; the higher the modulus of the

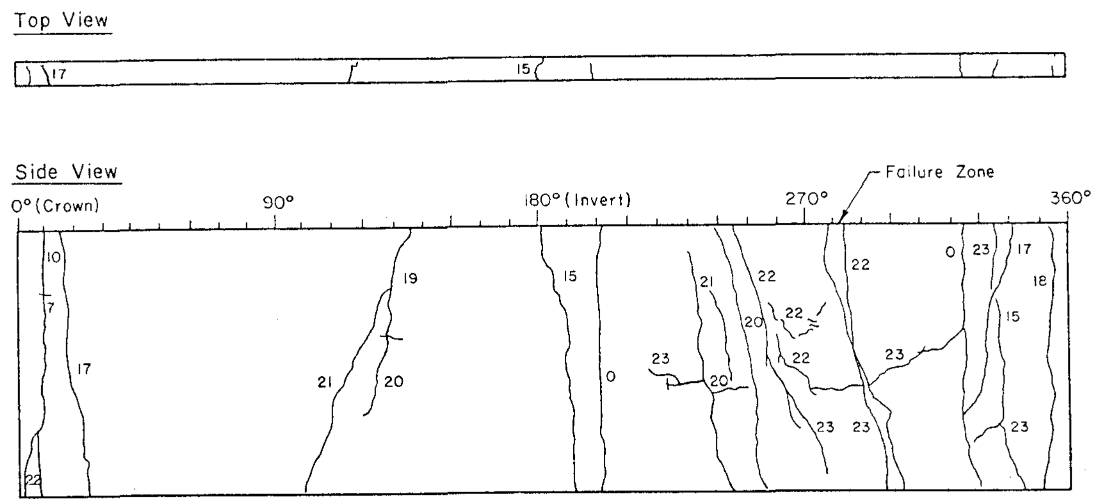


FIGURE 3.7 CRACKING PATTERN OF CIRCLE-1

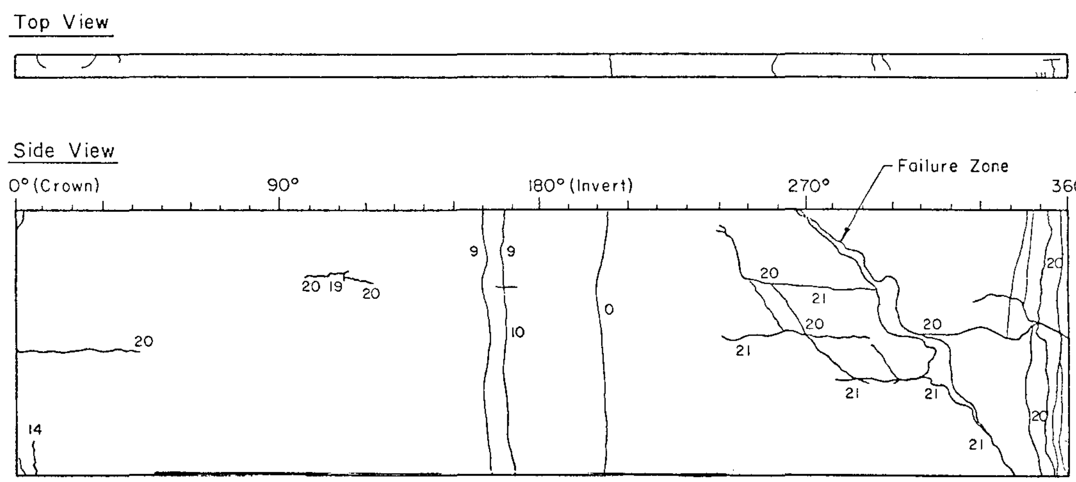


FIGURE 3.8 CRACKING PATTERN OF CIRCLE-2

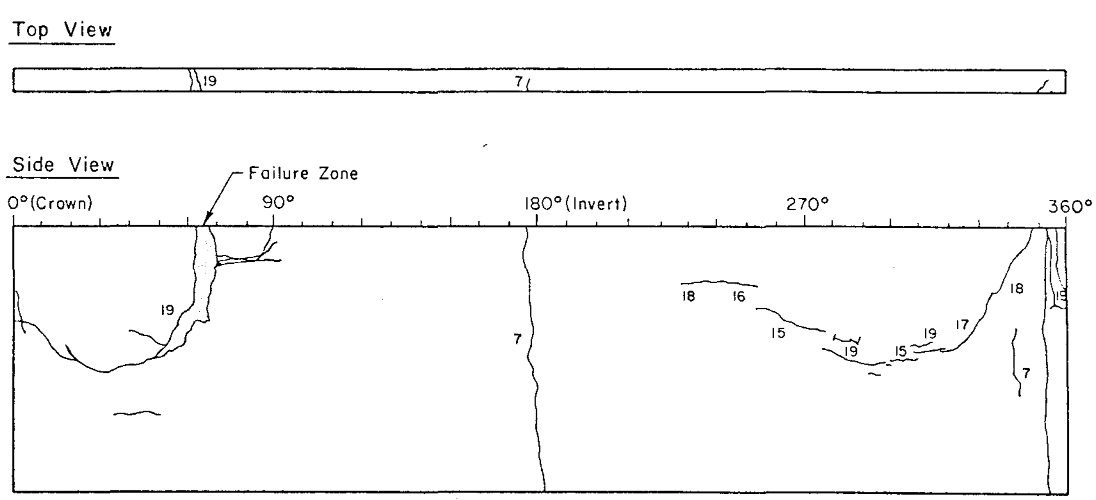


FIGURE 3.9 CRACKING PATTERN OF CIRCLE-3

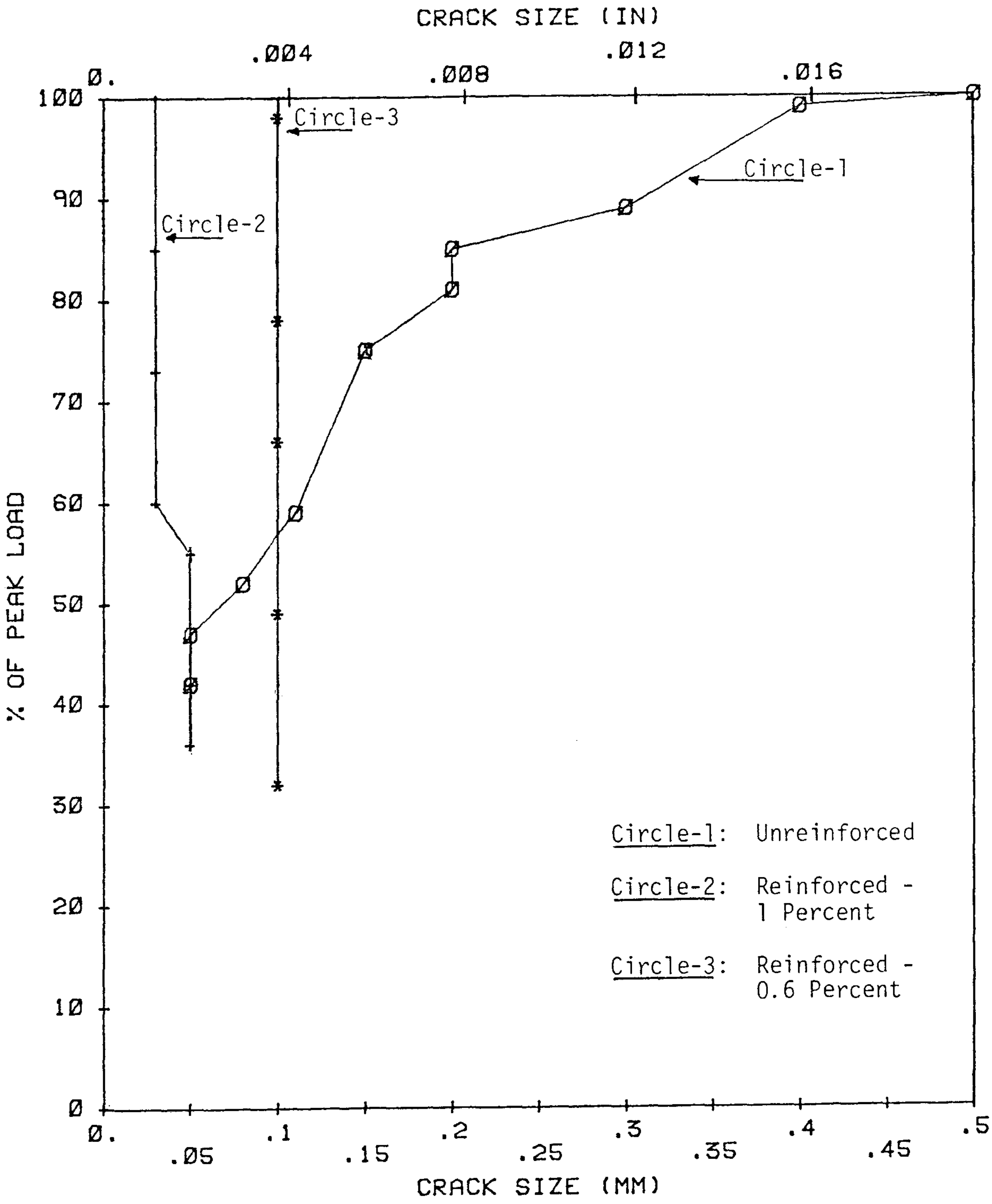


FIGURE 3.10 EFFECT OF AMOUNT OF REINFORCEMENT ON CROWN CRACK SIZE OF MONOLITHIC LININGS

medium, and thus the flexibility ratio, the higher the thrust ratio. Thus, preserving the integrity of the tunneled ground by a coordinated excavation and initial support sequence will not only reduce the loads that reach the final concrete lining, but it will also enhance its ability to support these loads if and when they occur. Furthermore, effective backpacking of the void between the concrete lining (cast-in-place or segmented) and the inside face of the tunneled opening is very important in ensuring the much needed passive resistance and proper redistribution of external loads and internal forces.

Effects of Joints (Monolithic vs Segmented Linings): Some indication of the effects of the joints on lining behavior may be obtained by comparing the load-deflection curves in Figure 3.11 of Circle-2 (monolithic) and Circle-5 (segmented) with the same medium modulus; the capacities of the linings are comparable, with the segmented lining exhibiting a slightly lower capacity due to the reduction of the cross-section at the joints.

The joints between the segments act as pre-formed cracks and thus by rotating during loading they decrease the moment in the lining. This in turn results in fewer cracks as shown in Figures 3.12 and 3.13. Furthermore, first cracks appeared at over 90 percent of the peak load for the segmented linings and their size was only about one-half of those in the monolithic linings. The effect of joints in reducing moments in the lining is magnified as the stiffness of the medium becomes lower, since the joint rotations increase, as observed by comparing the crack patterns of Circles-4 and 5. The changes in lining diameter are very close for the monolithic and segmented linings. In full scale linings the difference between segmented and monolithic may not be as dramatic as in these tests, because segmented linings are normally constructed with offset longitudinal joints between adjacent rings, and therefore the actual stiffness is between the values for a monolithic lining and that for a single ring of segments. The range of $\Delta D/D$ at 50 percent of the load is from 0.25 to 0.43 percent and at the peak load from 1.20 to 1.27 percent (Circle-5 excluded).

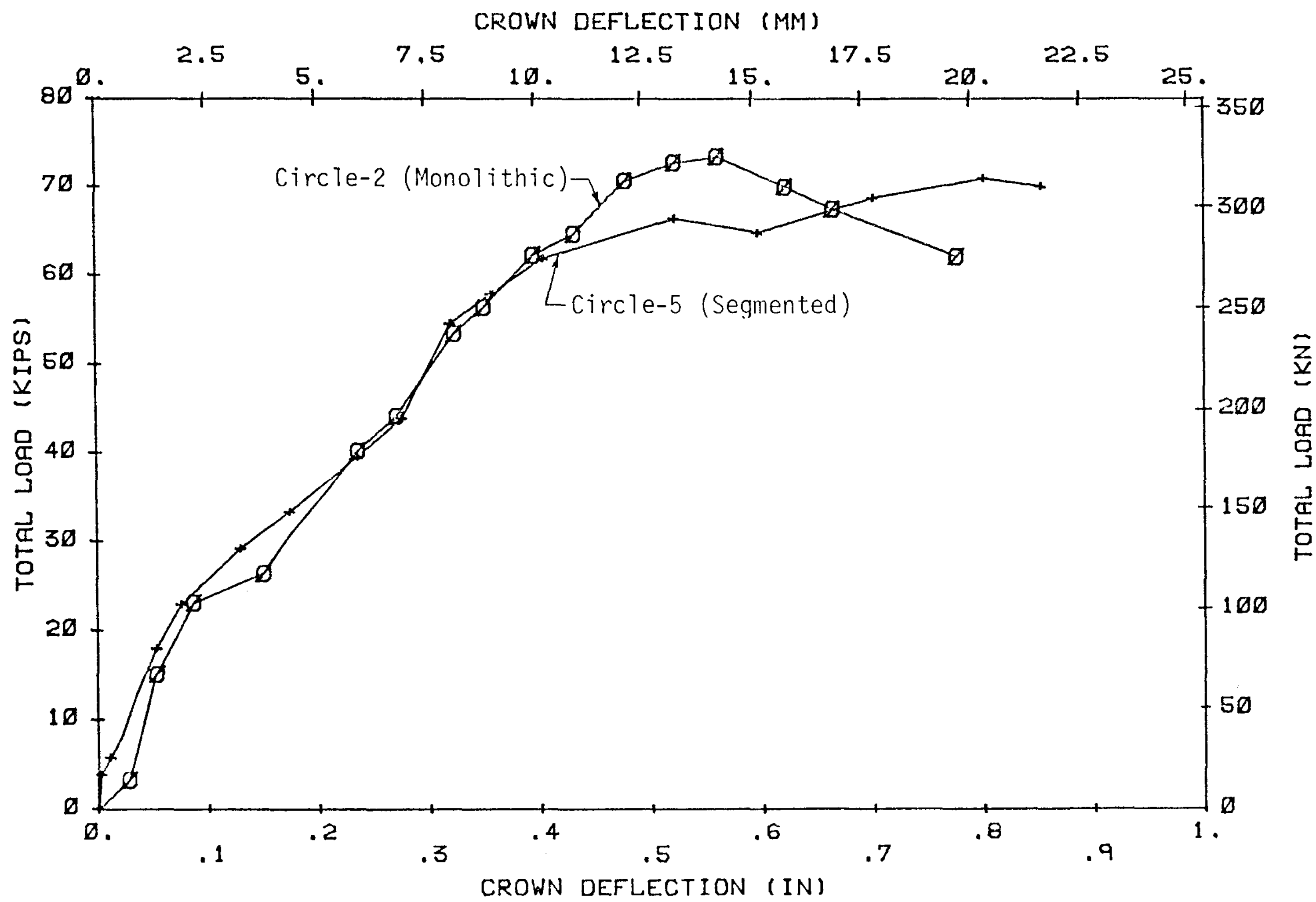


FIGURE 3.11 COMPARISON OF LOAD-DEFLECTION CURVE OF MONOLITHIC AND SEGMENTED MODELS CIRCLE-2 AND CIRCLE-5

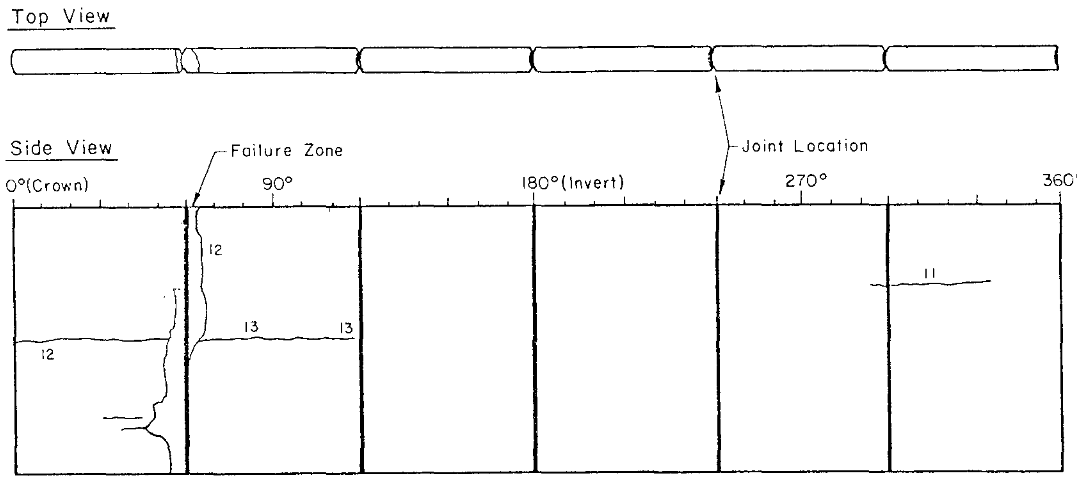


FIGURE 3.12 CRACKING PATTERN OF CIRCLE-4

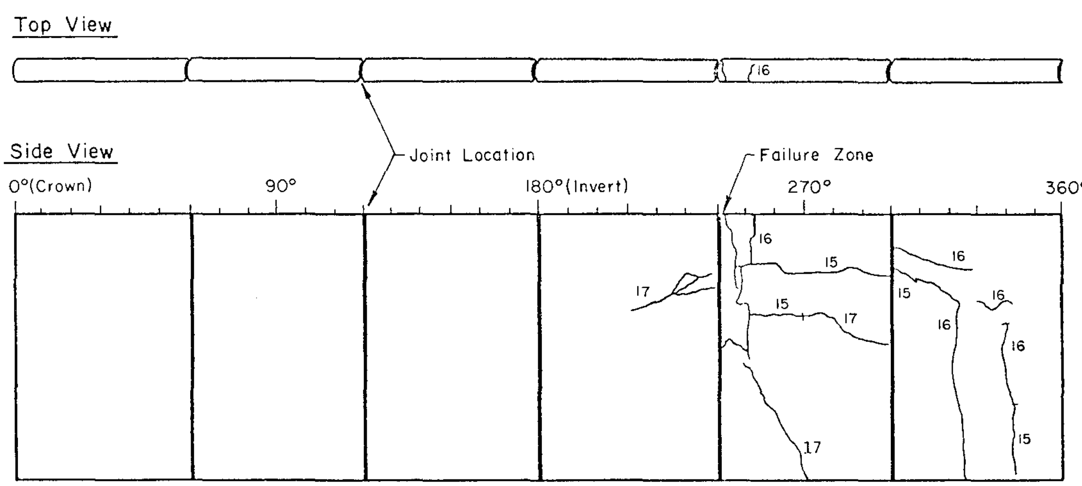


FIGURE 3.13 CRACKING PATTERN OF CIRCLE-5

Failure Modes: Failure of the linings occurred between 60 and 80 deg from the crown with the exception of Circle-5 where it occurred 120 deg from the crown at a joint. All the model lining failures resulted from crushing of concrete that began at the inside surface in some small region and with additional load it spread both through the depth of the section and longitudinally in the lining. The initiation of crushing was generally accompanied by cracking parallel to the direction of compressive stress (circumferential in the lining) typical of compression failures in concrete and to be expected in a specimen only one inch thick. Crushing began on one side of a joint in both segmented lining models. Circle-4 failed on both sides of the joint (Figure 3.12) while in Circle-5 crushing occurred only on one side (Figure 3.13). The joints constitute a weak section because of the reduced cross-section and the lack of reinforcement. There was no indication of high shear stress contributing to the initiation of failure or contributing to the spread of failed concrete after initiation (as might be indicated by a radial offset each side of the failure region if shear were a contributing factor).

3.2 ANALYTIC STUDIES

3.2.1 Parameter Study of Arches

Parameter studies were performed on semicircular arches with loosening loads using the beam-spring model to show the effect of flexibility ratio F, radius to thickness ratio R/t, ratio of tangential to radial stiffness of the springs K_t/K_r and load shape on a typical full scale station configuration. The model used is shown in Figure 3.14; radial springs in tension were inactive but all the tangential springs remained active. Nonlinear behavior of the lining was allowed due to material behavior and geometry change. A typical stress-strain curve for concrete was used. Radii of 20, 25 and 30 ft (6, 7.6, 9 m), thicknesses of 10, 12 and 15 in. (250, 300, 380 mm) and a concrete compressive strength of 4000 psi (27.6 MPa) were investigated. Most of the study was performed for a 12 in. (300 mm) thick arch of 25 ft (7.6 m) radius with a uniform load across the full arch and several problems

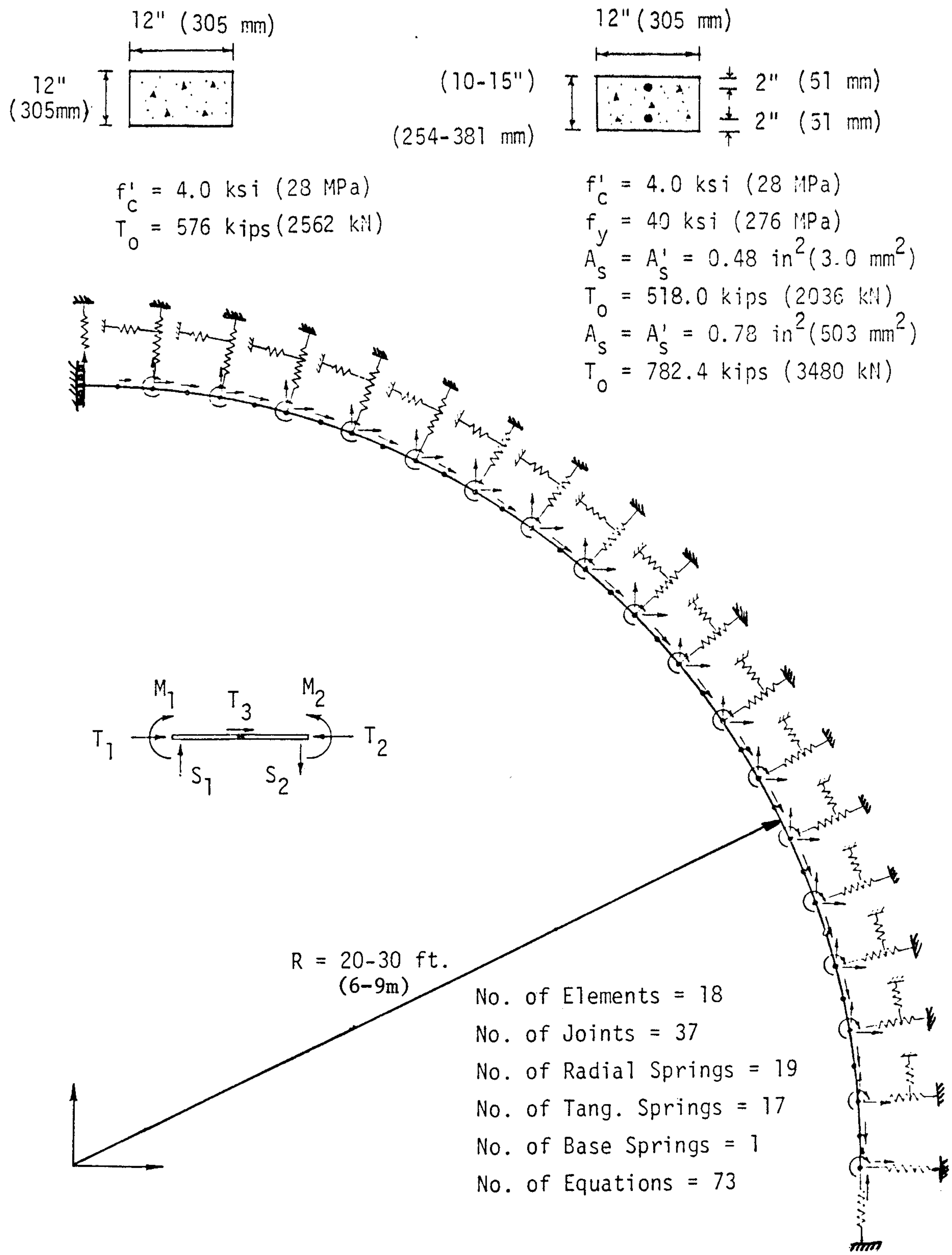


FIGURE 3.14 FINITE ELEMENT MODEL FOR PARAMETER STUDIES OF ARCHES

were worked with a symmetrical triangular load over the center 60 deg portion of the arch; to investigate further the effect of load shape one problem with uniform load over the 60 deg portion and one with uniform load over the right one-half of the arch were worked. The radius and thickness were varied in some problems to study their effects, while keeping the K_t/K_r ratio constant at 0.25. These solutions were for arch sections with one-half percent reinforcement in each face, and then a series of solutions were obtained for an unreinforced section for the full range of flexibility ratios and one value of tangential shear stiffness (K_t/K_r = 25 percent). As discussed in Section 3.1.1 the flexibility ratio F is used as a convenient measure of relative lining-medium properties even though it is derived for circular linings.

In Figure 3.15 the moment-thrust paths for the most critical section in the lining are shown on the interaction diagram for a flexibility ratio of 3500, lining thickness of 12 in. (300 mm), radius of 25 ft (7.6 m) and various load shapes. The symmetrical uniform load that covers the full span induces the smallest moment and therefore the largest thrust before failure. The triangular load over the central 60 deg arc induces the largest moment at the crown and consequently the smallest thrust and load at failure. The other two loading conditions fall between these two extremes with the unsymmetrical uniform load (over one half the arch) causing very nearly as much moment as the triangular load. However, reference to the geologic conditions that tend to cause various loads indicate maximum loads that can occur; if joint sets are assumed to occur in both directions relative to the vertical, a triangular wedge of rock depicted by the triangular load could occur about one radius wide at the base and have a height of about one-third diameter. The total weight per foot of tunnel for 50 ft (15.2 m) diameter would then be 31.3 kips (139 kN), if the rock is assumed to weigh 150 lb/ft^3 (2400 kg/m^3). The total load at failure from the analysis was 277 kips (1230 kN) so there is a safety factor of 8.8 against failure. If, however, the uniform load completely across the lining is assumed to represent a vertical depth of rock of one diameter, the total weight is 375 kips (1670 kN) per foot of tunnel.

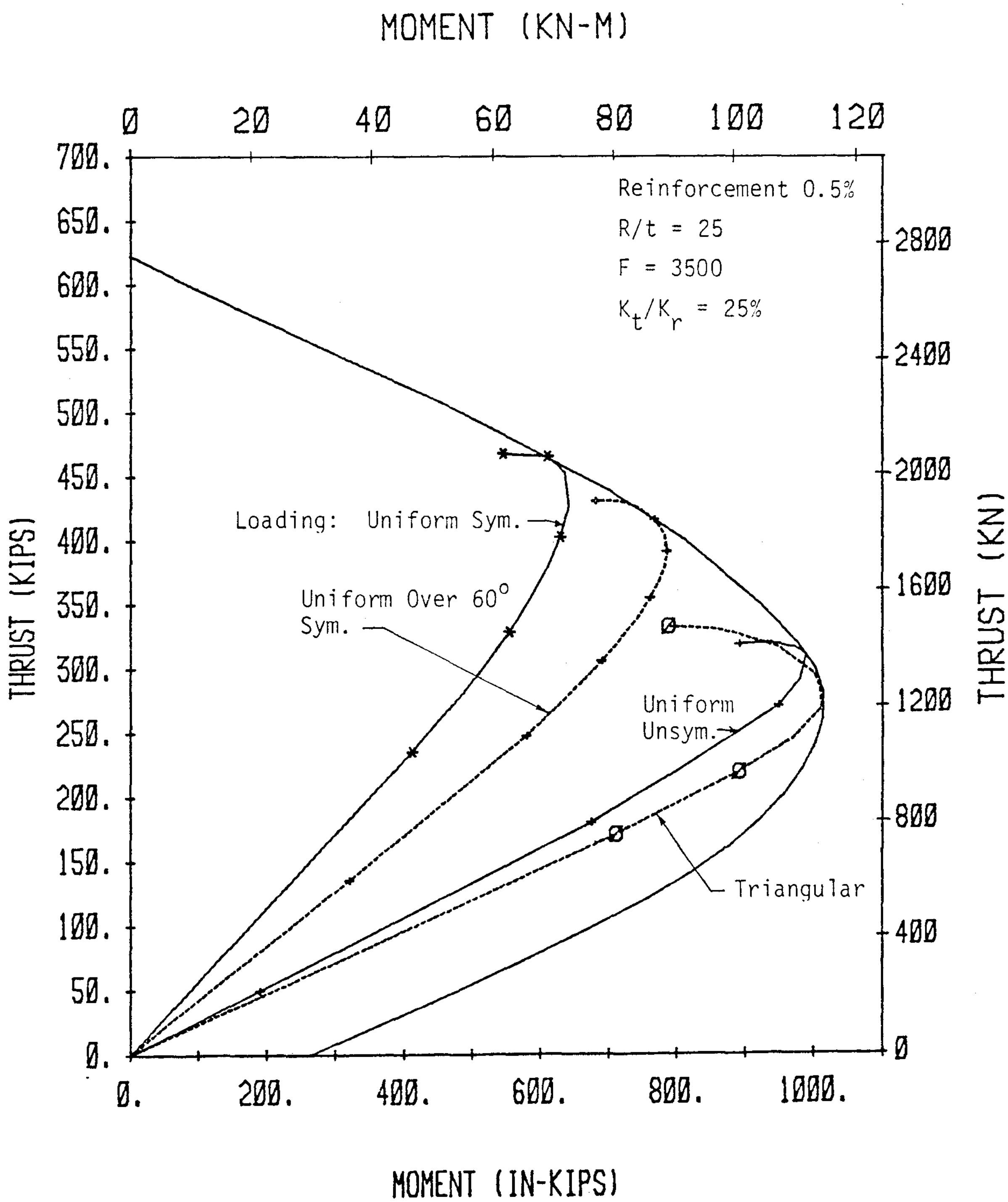


FIGURE 3.15 MOMENT-THRUST PATHS FOR CRITICAL SECTION FOR VARIOUS LOAD SHAPES

At failure the load was 992 kips (4420 kN) so the safety factor is 2.6 for this loading. This comparison shows that the triangular load is less critical though it produces more moment in the lining at a given thrust and thus results in failure at a smaller thrust and smaller total load. The safety factors obtained for partial uniform symmetrical and partial uniform unsymmetrical loadings were 5.0 and 4.8, respectively, which are still higher than the safety factors obtained for the uniform load across the arch.

Arbitrary dimensions of rock blocks have been assumed in the comparison above, and in reality they would depend on actual field conditions, but they are reasonable and show that the condition providing the greatest total load tends to be most severe. For all the symmetrical loadings the critical section occurred at the crown, and for the unsymmetrical one it occurred on the loaded side of the crown. These solutions were obtained for a lining thickness of only 12 in. (300 mm) which may be considered as about the lower limit of minimum constructable thickness of linings for such large openings. If the uniform rock load is not expected to be greater than one diameter above the crown, it may be concluded from the above analysis that the minimum constructable lining is adequate since the factor of safety for such loading is shown to be more than 2.

The effect of flexibility ratio F on total load on the lining for two loading shapes are shown in Figure 3.16 where the effect of K_t/K_r is also apparent. The vertical axis is changed to T_u/T_o in Figure 3.17, where the curve shapes are very similar because the total load and the ratio T_u/T_o are almost proportional for a given load shape. This proportionality is shown in Figure 3.18 and depends very little on the K_t/K_r ratio. With this in mind, Figure 3.17 can be considered the variation of load with F for different K_t/K_r ratios and load shapes. The curve is not as flat as that obtained from the model tests, but shows a definite decrease in sensitivity to F at larger values; in this range determining accurate values of rock and lining stiffness for calculating F are not as critical as it is in the low range where the curve is steep.

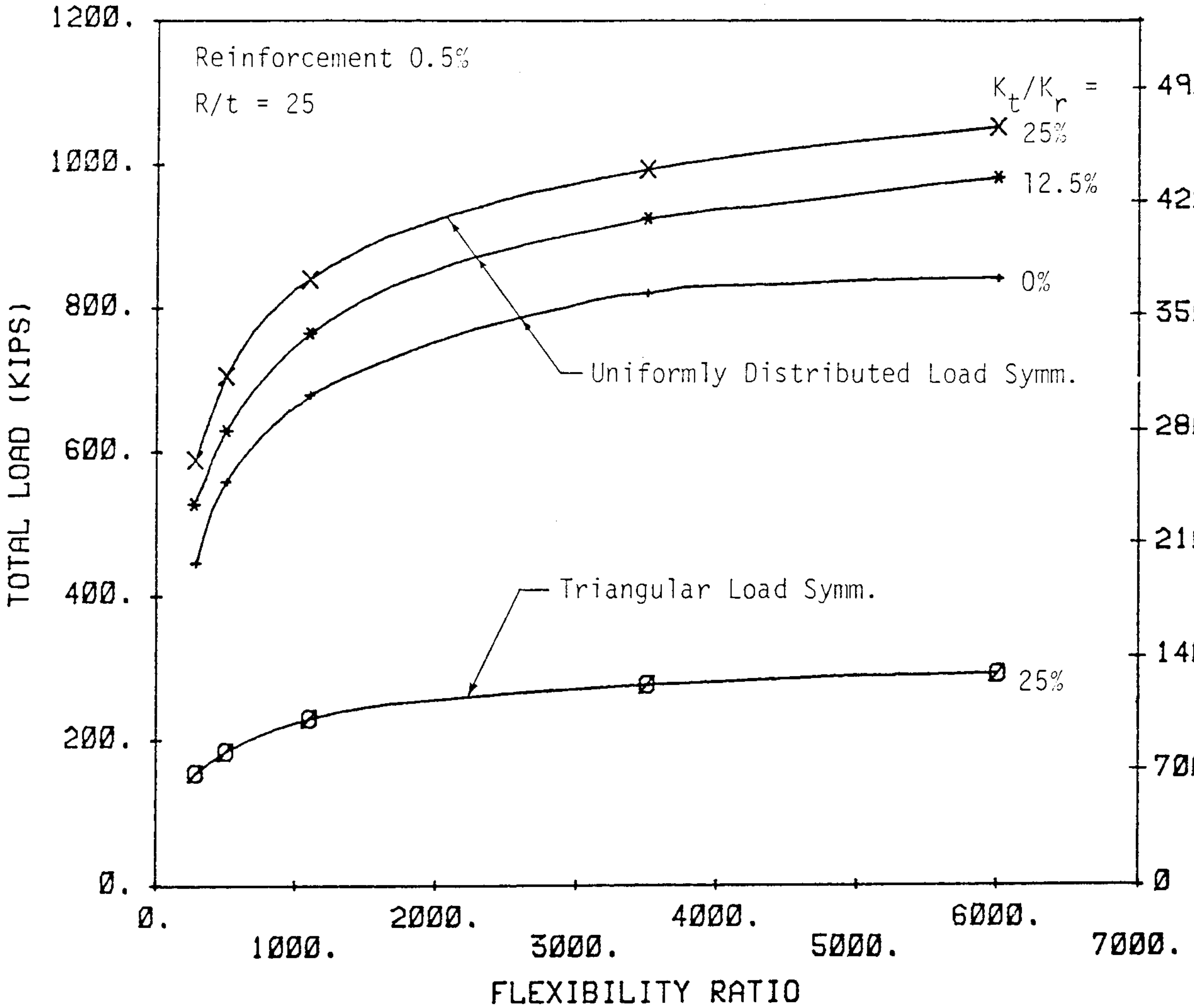


FIGURE 3.16 TOTAL LOAD VS FLEXIBILITY RATIO FOR ARCHES

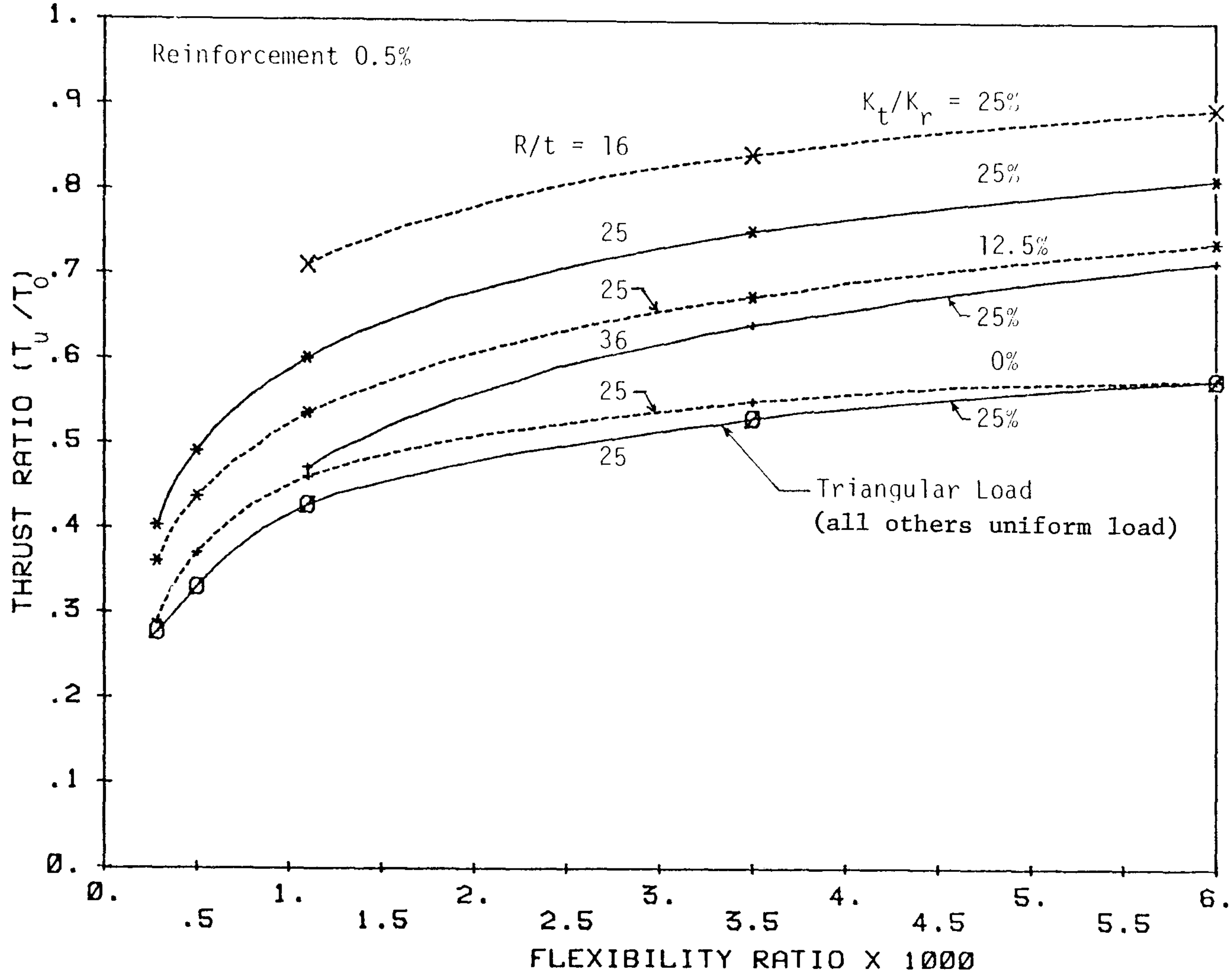


FIGURE 3.17 THRUST RATIO VS FLEXIBILITY RATIO FOR ARCHES

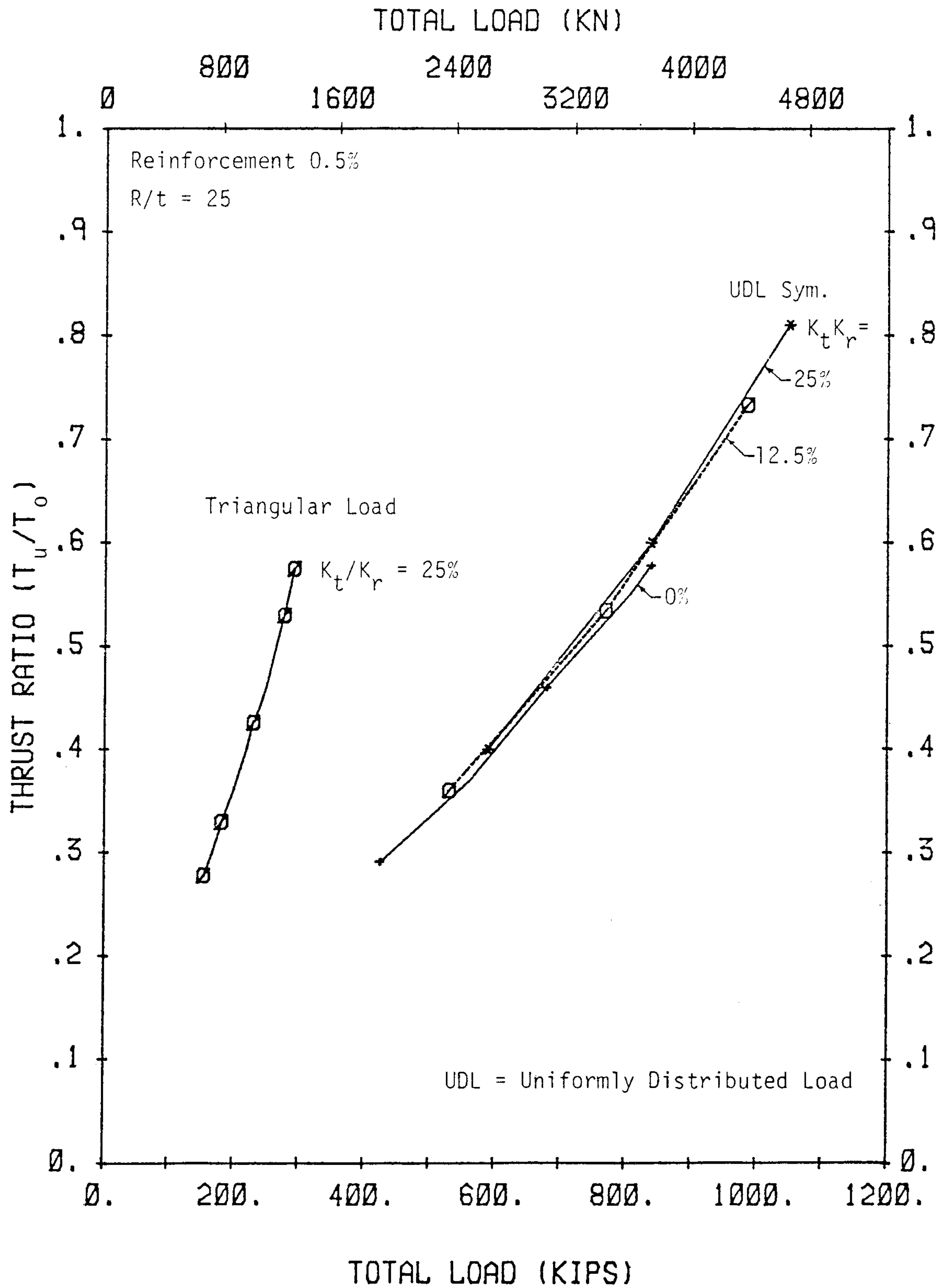


FIGURE 3.18 THRUST RATIO VS TOTAL ULTIMATE LOAD FOR ARCHES

The influence of F on the ratio of change in radius to radius ($\Delta R/R$) at failure in the direction of loading is shown in Figure 3.19. Linings in a soft medium (or low F) show considerably higher deformation, which decreases sharply as the flexibility ratio becomes larger. The deformations are also considerably higher for the case of no tangential shear than for the other values of K_t/K_r ratios.

Figure 3.20 shows the effect of tangential shear stiffness K_t/K_r on the moment-thrust behavior for the two extreme values of F equal to 6000 and 285, and the full uniform loading. For both values of F the curves are shown for K_t/K_r of 0, 0.125 and 0.25. For each value of F the radial spring stiffness remains the same, and the tangential stiffness changes. The moment-thrust paths reach the interaction diagram above the balance point for the larger F and below it for the smaller. For both values of F there is an increase in total load with K_t/K_r increase, and the tangential stress between the medium and lining can have a substantial effect on peak loads. Also, the absolute value of increase in load due to tangential shear increase is larger for the larger flexibility ratio, but the relative increase is smaller. The increase in load for an increment of K_t/K_r from 0 to 12.5 percent is greater than that for 12.5 to 25 percent. This effect is more pronounced for large flexibility ratio, as shown also in Figure 3.21 where T_u/T_o is plotted against K_t/K_r.

A lining with the same dimensions and concrete stress-strain curve was investigated without reinforcement. The ultimate pure thrust capacity (T_o) is reduced to 576 kips (2560 kN). The moment-thrust envelope for the unreinforced lining is also smaller than the reinforced one in Figure 3.22 where the moment-thrust paths for different flexibility ratios and for $K_t/K_r = 0.25$ and $R/t = 25$ are shown. The moment in the unreinforced lining is slightly smaller at a given thrust than that in the reinforced one because removal of the reinforcement decreased the lining stiffness and therefore increased F, and an increase in F causes a decrease in moment. However, this difference in the initial moment-thrust ratio is small. As the moment-thrust paths approach their respective failure envelopes, the

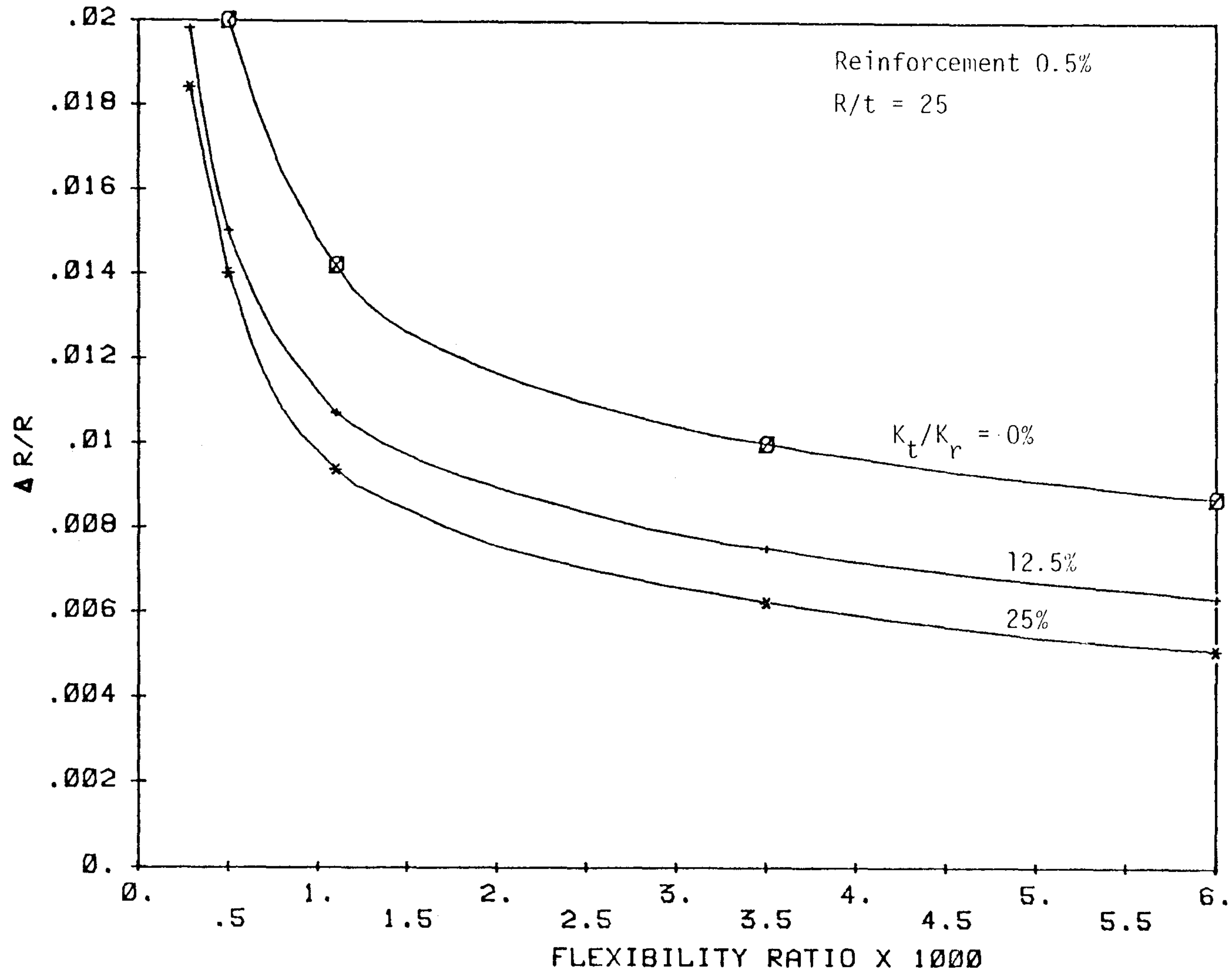


FIGURE 3.19 VARIATION OF RADIUS CHANGE RATIO AT FAILURE WITH FLEXIBILITY RATIO

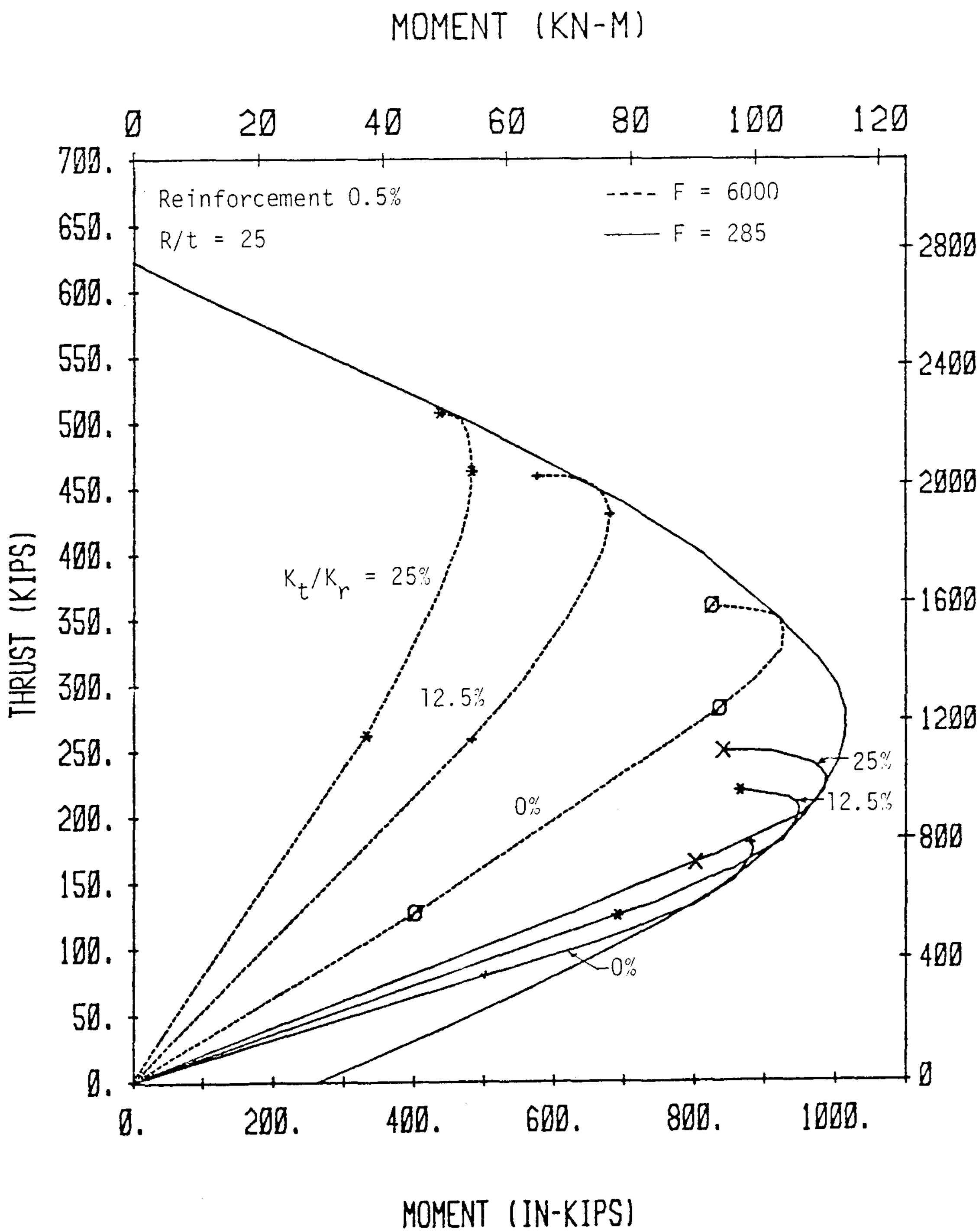


FIGURE 3.20 EFFECT OF K_t/K_r ON MOMENT-THRUST PATHS FOR ARCHES

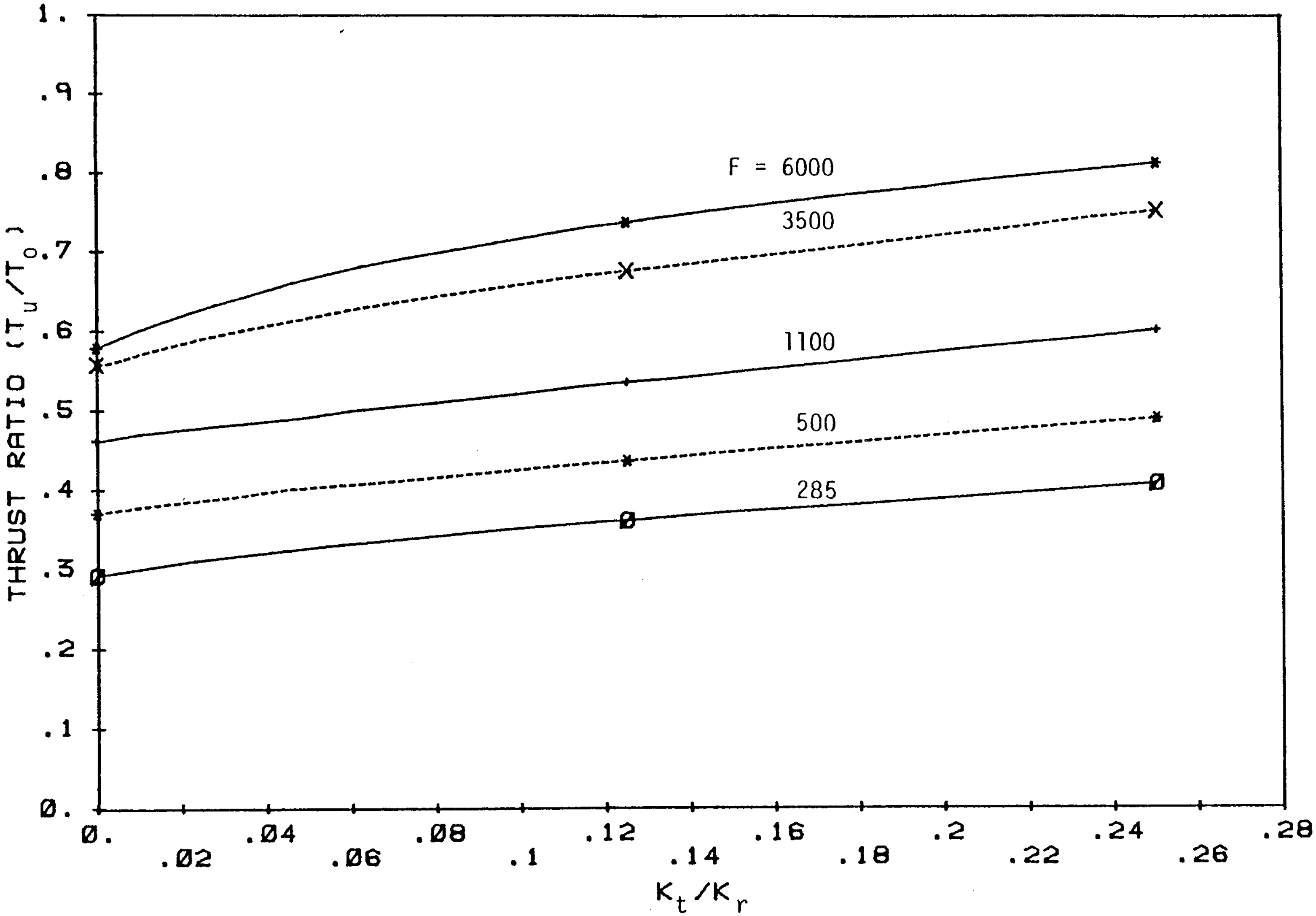


FIGURE 3.21 EFFECT OF K_t/K_r ON THRUST RATIO FOR ARCHES

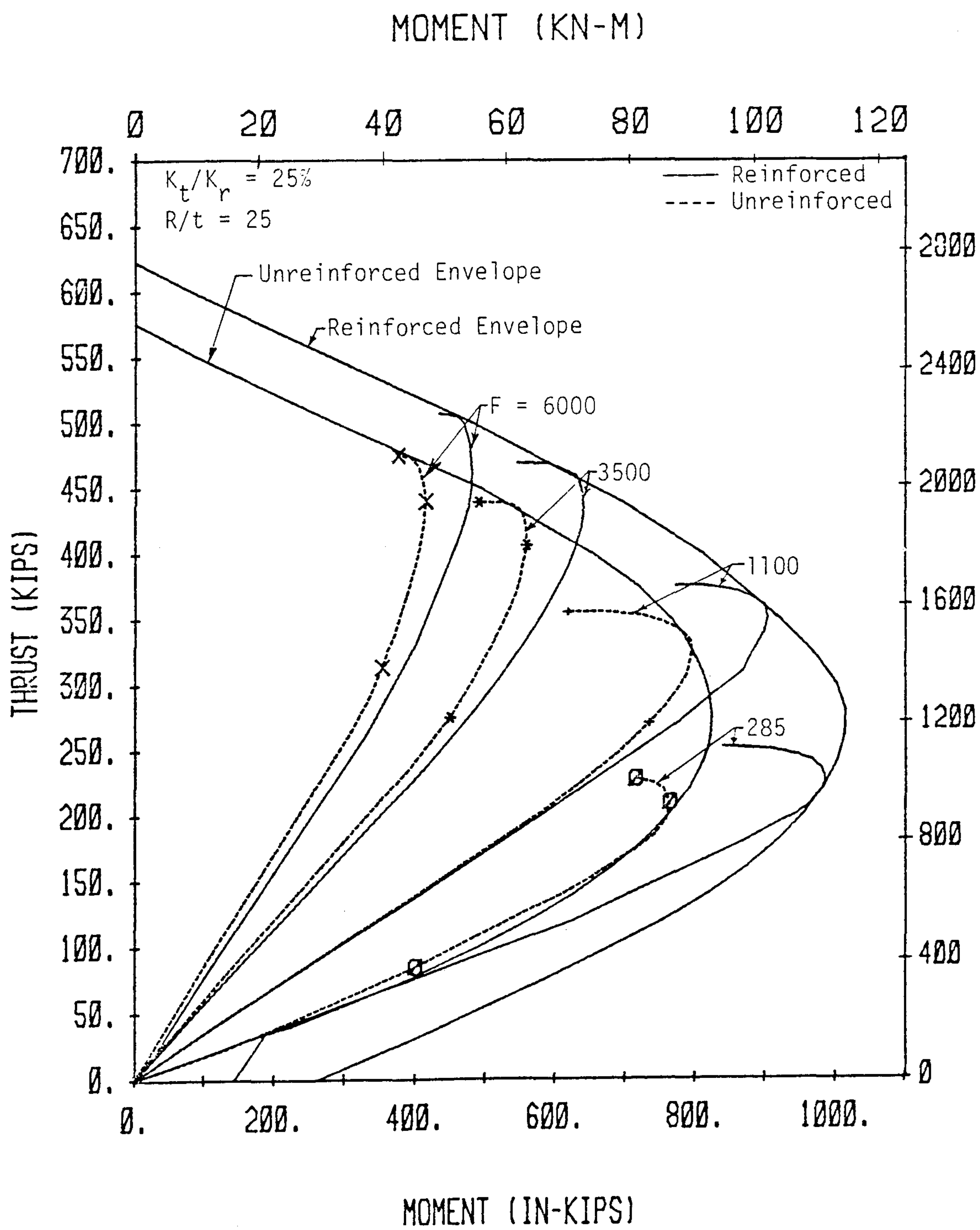


FIGURE 3.22 EFFECT OF REINFORCEMENT ON MOMENT-THRUST PATHS FOR ARCHES

ultimate thrust (T_u) and correspondingly the maximum load for the reinforced lining is higher than the unreinforced lining because the envelope is higher. However, the ratio T_u/T_o increases for the unreinforced lining by a small amount as shown in Figure 3.23. This difference in T_u/T_o for the two cases would be reduced, however, if the actual values of F were computed for the reinforced section based on the transformed section.

Solutions were obtained for values of radius to thickness ratio of 16, 25 and 36 based on radii from 20 to 30 ft (6 to 9 m) and thicknesses from 10 to 15 in. (250 to 380 mm). The effect of the R/t ratio was studied by keeping the K_t/K_r at 25 percent, the reinforcement ratio at 0.5 percent in each face and varying the dimensions. Figure 3.17 shows that as the R/t ratio increases, thrust ratio decreases when F and K_t/K_r are kept constant. The variation of R/t from 16 to 25 and from 25 to 36 renders similar changes in thrust ratio. Thus linear interpolation is possible for intermediate values of R/t and the same F and K_t/K_r.

In summary, the parameter study indicates that a uniform loading across the entire arch has the smallest safety factor against collapse for the particular conditions investigated. Though a triangular and unsymmetrical uniform loading provide lower values of T_u/T_o, they still have larger safety factors for the rock block dimensions selected. This study was not broad in its coverage of parameters, however, and the unsymmetrical loadings should be investigated at a particular site.

The studies also show that strength increases with F though there is a definite flattening of the curve in Figure 3.17 above F = 1000, indicating less sensitivity to the calculation of F. An increase in shear stress between the lining and medium results in a definite increase in strength of the lining. Therefore, it is essential to include the shear stress in the analysis of the lining to obtain a realistic prediction of strength, especially for larger values of F, but the strength is not highly sensitive to K_t/K_r as shown by the flatness of the curves in Figure 3.21.

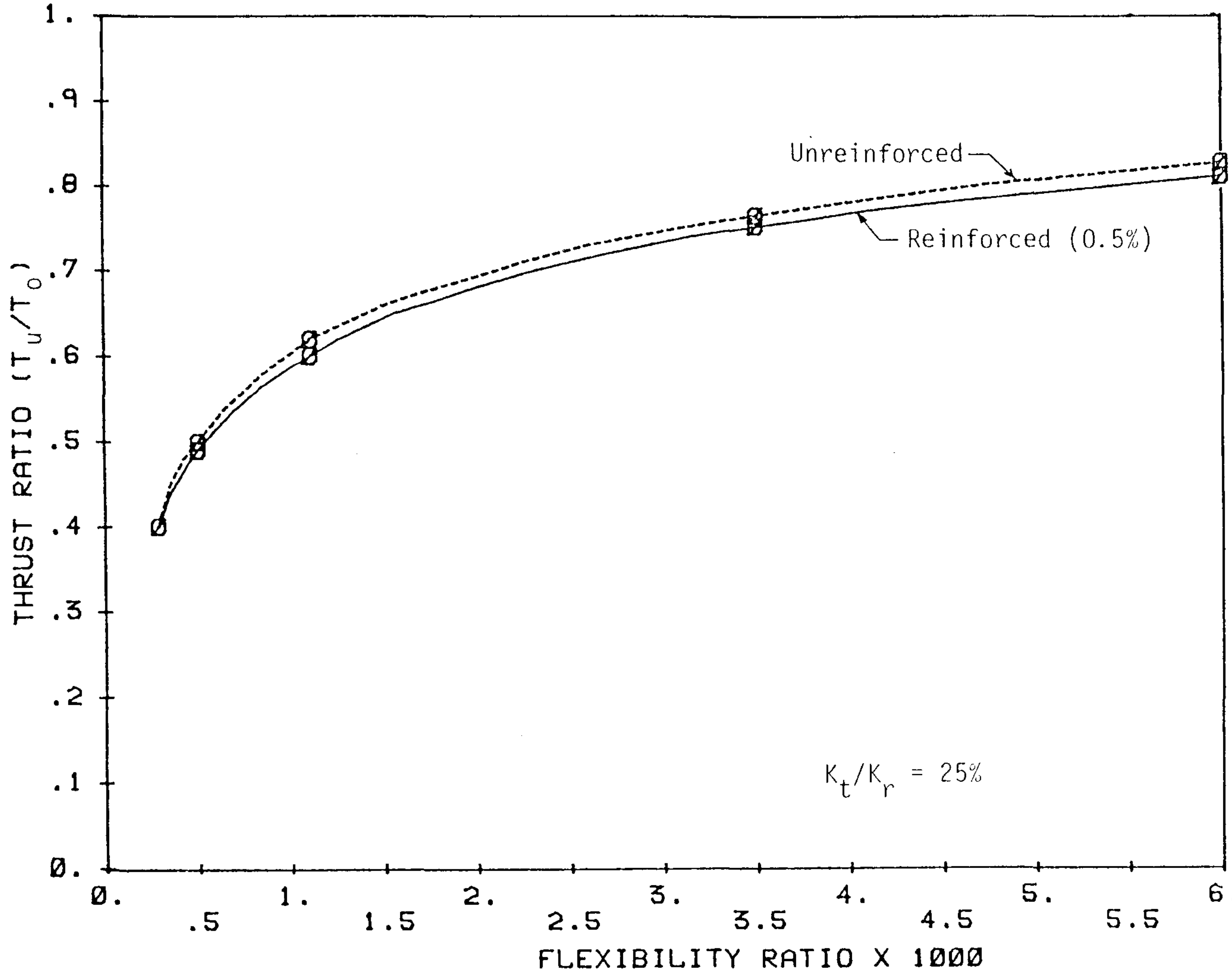


FIGURE 3.23 EFFECT OF REINFORCEMENT ON THRUST RATIO FOR ARCHES

Reinforcement increases the absolute strength approximately as would be indicated by an analysis of the section, but both the model tests and parameter studies show that the effect on the T_u/T_o ratio is very small. That is, T_u and T_o increase in the same ratio, so the reinforcement has little effect as might result from changing the relative stiffness of the lining and medium.

3.2.2 Parameter Studies of Circular Linings in Soft Ground

Description of the Program: An existing nonlinear finite element program that used springs to represent the medium was modified to provide a better representation of soft-ground conditions. The existing program used a special three-node beam element to represent the lining that can model reinforced or unreinforced concrete sections with nonlinear stress-strain properties of the concrete and reinforcement. This spring representation for the medium does not properly represent all the interaction components that occur when a lining is placed in a continuous homogeneous medium because the load must be applied directly to the lining, and it does not account for arching of loads around the lining. Hence for better representation of the medium as well as to be able to handle various other loading conditions, the program was modified by incorporating two dimensional 8-node quadratic isoparametric elements that will represent the medium as a continuum. Also a special type of interface element was devised to represent slip at the ground-lining interface.

For efficient handling of large problems, the solution method was modified to use a multiple-level substructuring scheme, so the medium could be divided into a number of linear substructures and static condensation performed for each one to compute condensed stiffness matrices and equivalent load vectors. This process of substructuring could be continued sequentially at subsequent higher levels reducing the size of the total structural stiffness matrix for the medium to a size that can be handled by the computer. This scheme also allows the use of a particular substructure representation of the medium repeatedly in several problems with the same medium geometry but different elastic moduli without recomputing the stiffness matrix. An interface element was devised, (Saha, 1982), that is placed at each node and oriented tangent to the lining curvature at the joint. This element has been modeled with two relative displacement degrees of freedom, one in the normal and the other in the tangential direction, and zero stiffness is assigned to the elements when separation occurs. For the purpose of parametric studies, only nondilatant joint properties were given. Very high normal stiffness is assigned to model contact and a Mohr-Coulomb criteria is used to model elasto-plastic tangential shear deformation at the interface.

Description of the Study: In the analysis of linings in soft ground the greatest uncertainty lies in the way the ground loads reach the lining. Some of the commonly used loading conditions are described in Section 1.2.3 and are termed i) overpressure loading, ii) excavation loading, and iii) gravity loading. In the parametric studies the lining behavior under these loading conditions were studied and compared in view of the degree of their severity at both ultimate and first cracking levels. Linear closed form solutions for the first two types of loading were also solved for a wide range of parameters and compared with the corresponding linear and nonlinear finite element solutions using the modified program. The comparison of the linear solutions also served to verify the program, while that with the nonlinear solution resulted in an evaluation of the effect of nonlinearity of the problem in terms of redistribution of internal forces and additional strength over that predicted by linear analyses. Loading conditions consisting of water pressure or removal of internal air pressure were also investigated. Other variables considered were the interface properties such as cohesion c and internal frictional angle ϕ, shear modulus G, coefficient of earth pressure K_o, and reinforcement. The effects of these parameters on the behavior of the lining are presented briefly in the following sections.

Effect of Interface Properties: The slip condition at the ground-lining interface is controlled by three parameters defining the material properties for the interface element: cohesion c, angle of internal friction ϕ, and shear modulus of the medium G. Variation of cohesion within reasonable limits (0 to 5 psi) did not have an appreciable effect on the solutions; there is very little difference in the crown deflection, tangential and radial pressure distribution, or moment and thrust distributions as shown in Figures 3.24 and 3.25. For this reason a small value of c has been used in most of the problems for parametric studies as it facilitates obtaining convergence. Variation of the angle of internal friction did show an appreciable effect; with high ϕ (= 45 deg), the shear strength of the interface increases and larger tangential shear stresses result, which reduces the thrust at the crown and invert. The thrust distribution for a small ϕ remains fairly uniform since the tangential shear strength remains low at the interface, which leads to a condition near full slip as most of the interface elements become plastic in shear at this load level. On the other hand for higher ϕ, a partial slip or nearly no slip situation arises. Moments are larger for low ϕ and since crown thrust is lower, the critical section is at the crown for low flexibility ratios ($F < 7$). However, the total load does not differ appreciably between full slip and partial slip or no slip conditions as shown in Figure 3.26. On the other hand for higher flexibility ratios ($F > 7$), the thrust at the springline becomes so much higher for the partial slip case that the failure section changes to the springline and failure occurs at a lower load than in the full slip case. Thus the load capacity is lower for the partial or no slip case and the difference increases with the increase in flexibility ratio (Figure 3.26). The same trend occurs for excavation loading also.

Effect of Water Pressure: For studying the effect of water pressure on the lining behavior and on the total load capacity, uniform all-around water pressure was added to the ground loads in the incremental solution process until an estimated service load was reached and then the incremental ground loading was continued. The moment-thrust paths shown in Figure 3.27 for three different water pressures (0, 2.5 $\gamma_w D$ and 5 $\gamma_w D$ where γ_w is the unit weight of water and D is the lining diameter), clearly indicates the benefi effects of water pressure. Since a uniform pressure does not induce moment, only

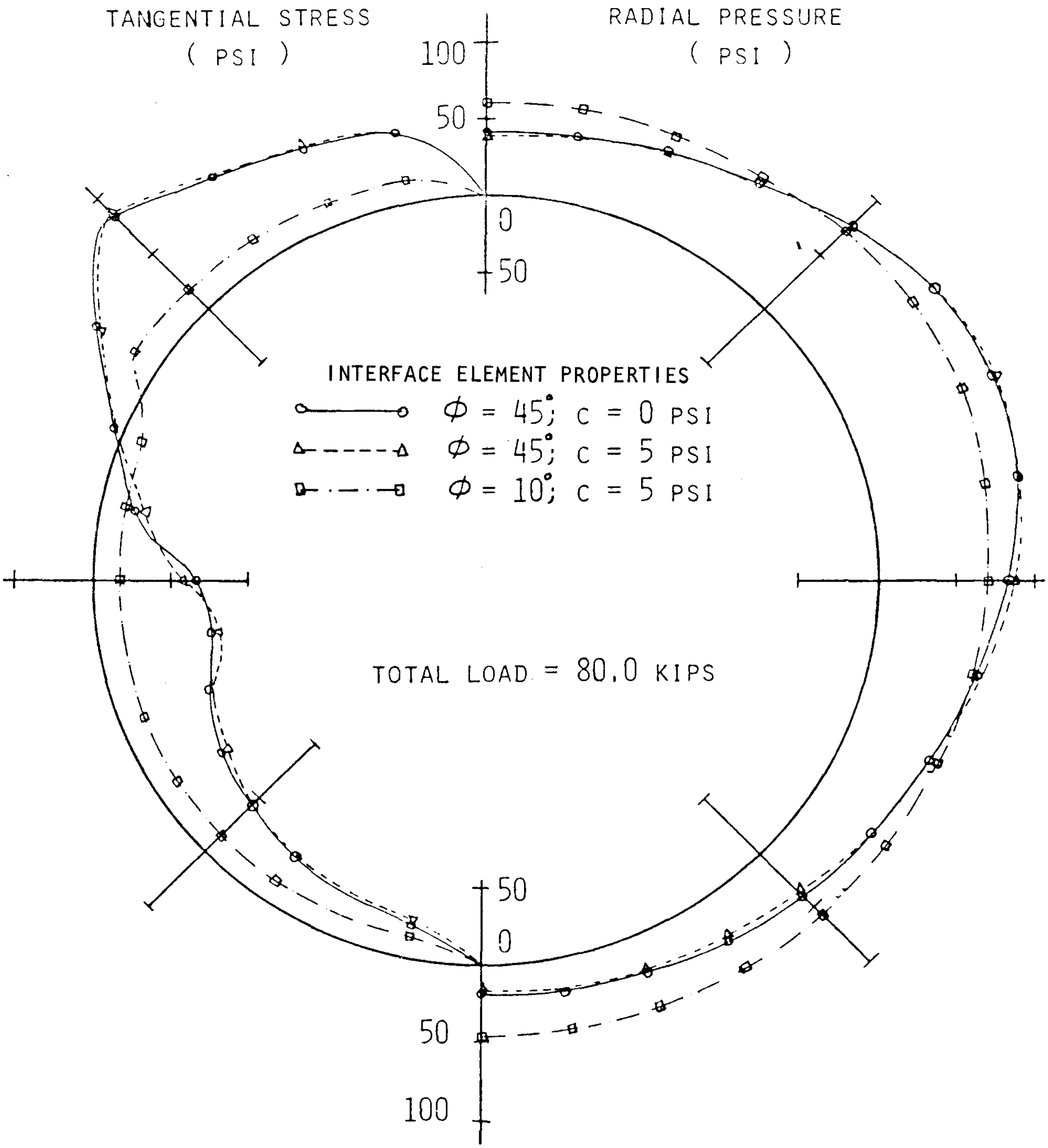


FIGURE 3.24 EFFECT OF ϕ AND c ON RADIAL PRESSURE AND TANGENTIAL STRESS

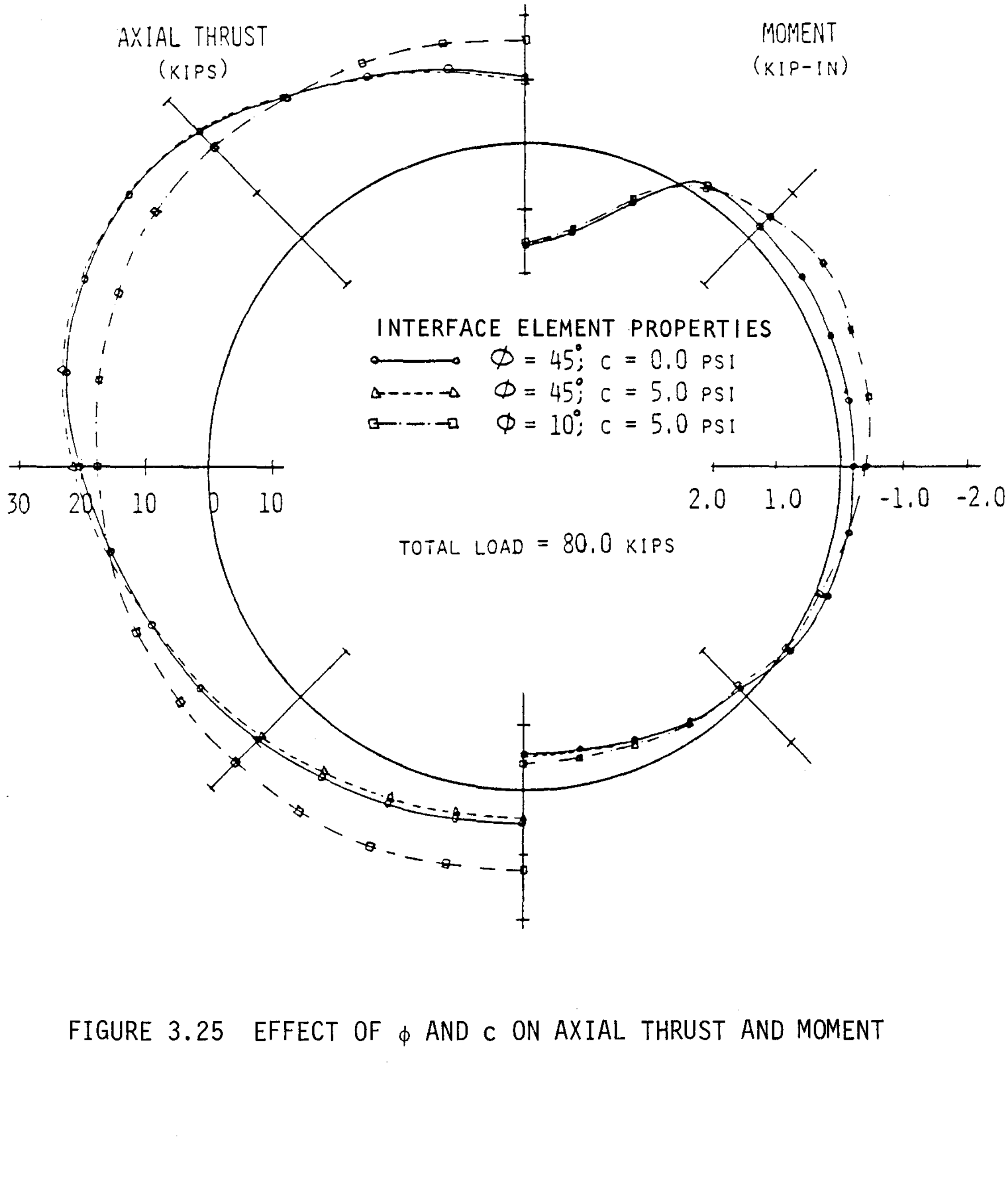


FIGURE 3.25 EFFECT OF ϕ AND c ON AXIAL THRUST AND MOMENT

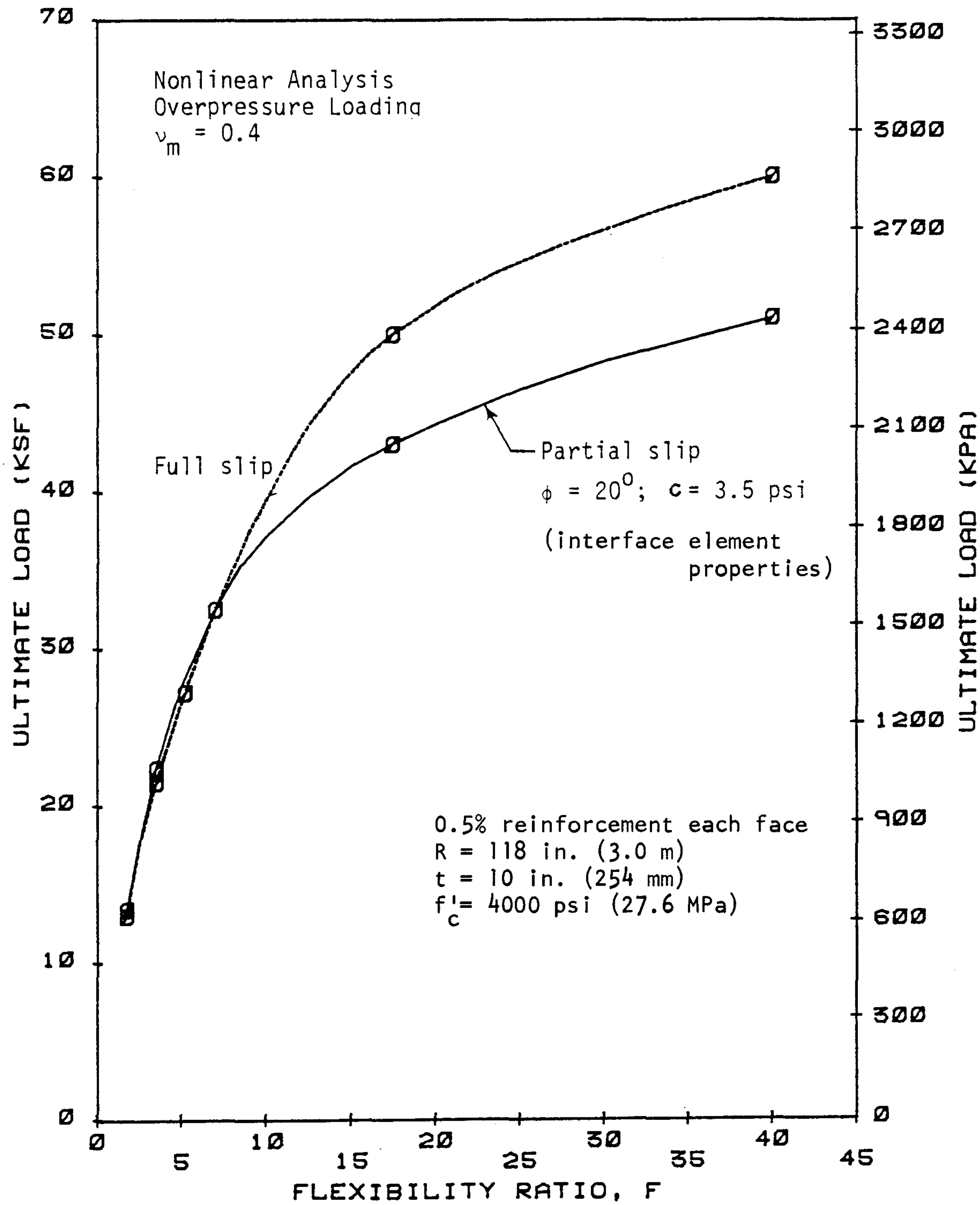


FIGURE 3.26 EFFECT OF SLIPPAGE ON ULTIMATE LOAD

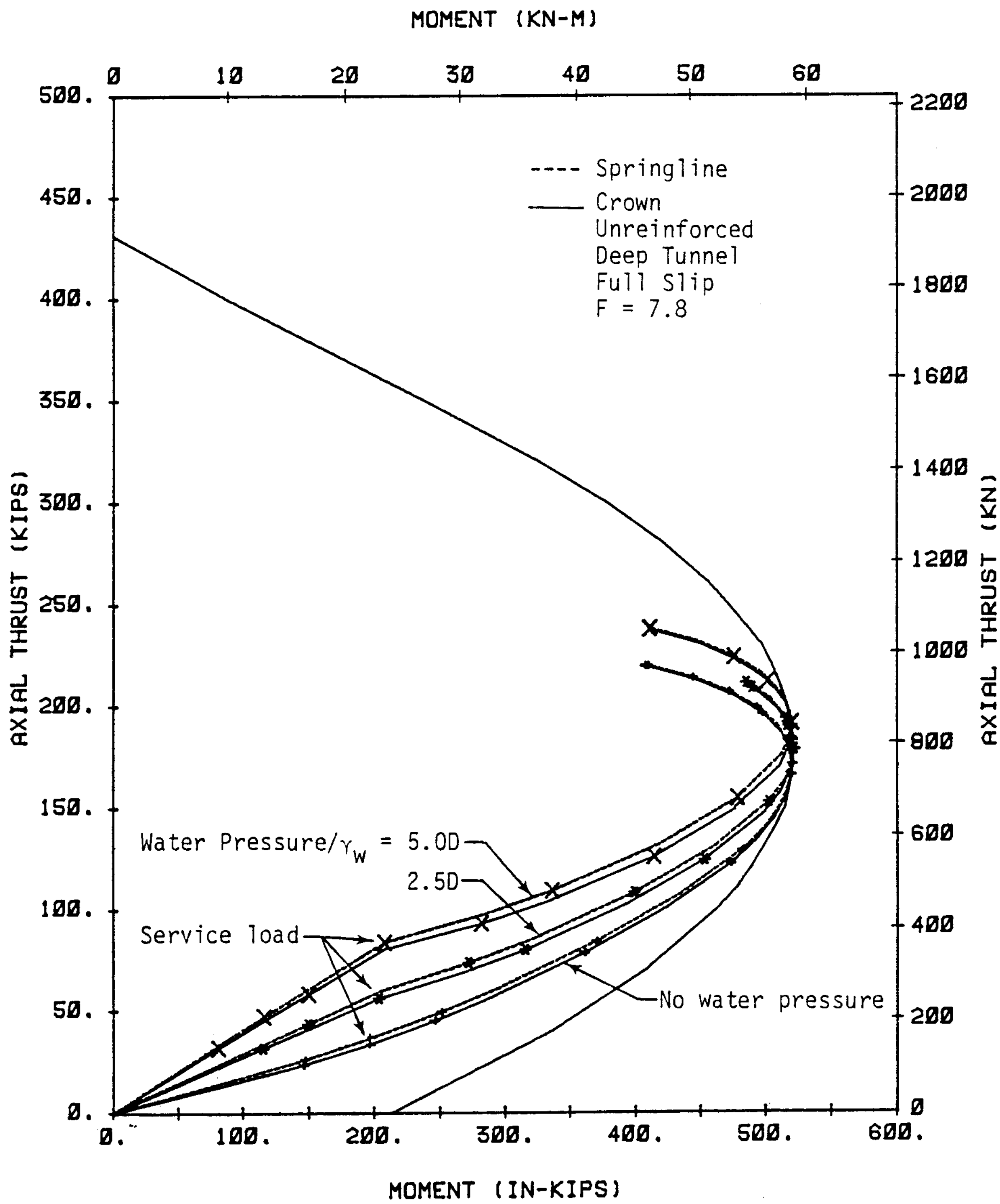


FIGURE 3.27 EFFECT OF WATER PRESSURE ON MOMENT-THRUST PATHS

the thrust is increased with the increase in water pressure until service load. This reduces the slope of the moment-thrust paths (eccentricity) which helps in avoiding or reducing cracks in the lining. Table 3.4 summarizes the effect of water pressure on various other parameters and indicates that although the thrust ratio increases with the increase in water pressure, total ultimate vertical load is not changed appreciably.

Effect of Loading Conditions: Three different loading conditions were investigated to determine their effect on the lining behavior under different conditions of slip for several flexibility ratios. The overpressure loading condition was modeled by applying uniform pressure at the top surface of the initially unstressed medium with the lining in place. The excavation loading condition, explained in Section 1.2.3, was obtained by applying the shape of the in situ stresses at the interface on the medium nodes and increasing the magnitude proportionally. The excavation and overpressure loading were applied to a lining in a linear medium with joint elements between the lining and medium having properties corresponding to an angle of internal friction ϕ of 20 deg and cohesion of 3.5 psi (0.024 MPa). The beam-spring model was used for the gravity loading cases. A wide range of variables and loadings were investigated for these loadings conditions for a 10 in. (254 mm) thick lining with 118 in. (3.0 m) radius and 0.5 percent reinforcement in each face. For selected cases for the gravity loadings, the problems were also run for the same lining but without reinforcement. The compressive strength of the concrete was 4000 psi (27.6 MPa).

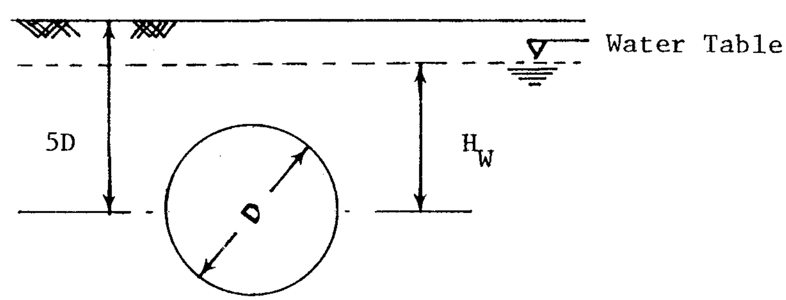
The difference in capacity between the overpressure loading and excavation loading increases with flexibility ratio up to a flexibility ratio of about 20, and then remains fairly constant as shown in Figure 3.28. The overpressure loading will generally lead to an overly conservative design and the excavation loading is more suitable.

Another important parameter in connection with the excavation loading is the coefficient of earth pressure K_o. For very soft soil

TABLE 3.4 EFFECT OF WATER PRESSURE ON VARIOUS PARAMETERS

H_w[1]	Total Ultimate Load ksf	Factor of Safety	T_u/T_o	$\Delta D_{cr}/D$ at Ultimate	ε_{sp} at Ultimate	$\Delta D/D$ at Service Load	Vertical Load at Cracking ksf
0.	22.8	5.36	.419	2.0	.0048	.313	2.394
2.5D	21.4	5.02	.449	1.879	.0045	.317	3.098
5 D	20.0	4.69	.478	1.916	.0045	.325	4.26
Effect of Increase in Water Pr.	Not Appreciable Change	Not Appreciable Change	Increases	Not Appreciable Change	---	Not Appreciable Change	Increases

[1] H_w is defined as below:



$F = 7.85$; $E_m = 10$ ksi; $D = 17.75$ ft.; $t = 9$ in. unreinforced; $f'_c = 4000$ psi; γ_s = unit weight of soil = 120 lbs/ft^3; service load = $2D \cdot \gamma_s = 4.26$ ksf

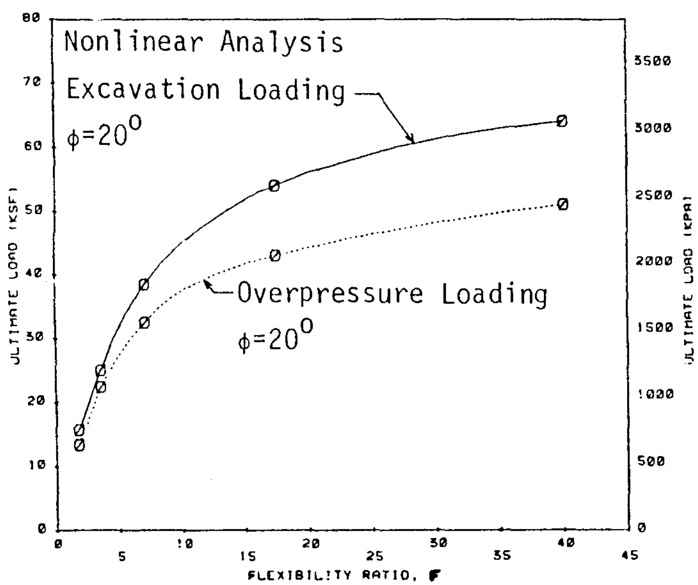


FIGURE 3.28 EFFECT OF LOADING CONDITION ON TOTAL ULTIMATE LOAD

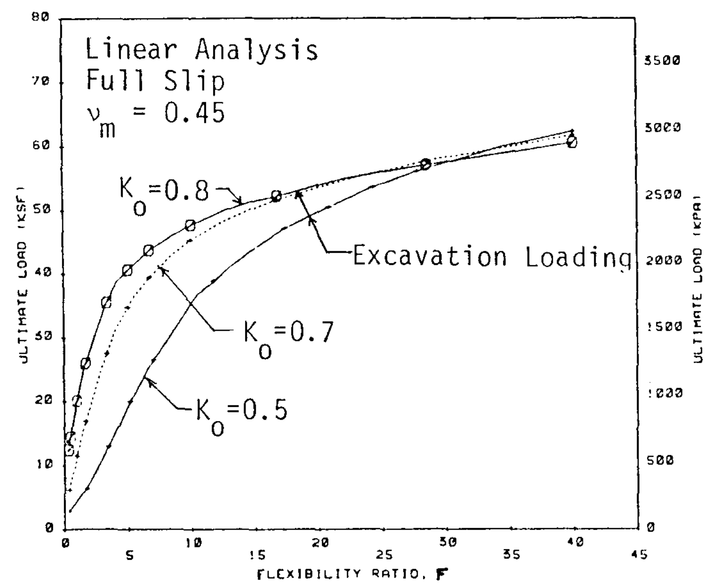


FIGURE 3.29 EFFECT OF COEFFICIENT OF EARTH PRESSURE ON TOTAL LOAD

this parameter could be near 1.0, while for firmer soils it may be on the order of 0.5. The effect of the variation of K_o on the total load is shown in Figure 3.29; the strength in each case is obtained by using a linear analysis and obtaining the ultimate load by projecting the moment-thrust path to intersect the interaction diagram. At low flexibility ratios the ultimate load is increased greatly by increasing K_o, but the difference decreases as K_o becomes large until the curves cross at a flexibility ratio of 30. Therefore it is important to use a realistic value of K_o, especially in the range of flexibility ratios from zero to about 20.

Some of the gravity loading cases are quite severe as a result of large moment at the crown while the corresponding thrust is small, because horizontal forces result only from passive resistant due to horizontal ovaling. The manner in which the gravity forces act on the lining in any particular case, depends on the soil condition and the excavation and support procedure. Several loading shapes were investigated with the beam-spring model while varying the tangential spring stiffness relative to the radial values between zero and 40 percent. It is believed that the case with full slip or no shear stress between the lining and medium is far too conservative for this loading case and that a significant shear stress would always be present. Four load shapes were considered: (1) a uniform vertical load across the full lining, (2) only the radial components of a vertical uniform load across the full lining acting on the lining, (3) uniform radial load around the upper 180 deg of the lining and (4) radial load over the upper 60 deg segment centered at the crown (See Figure 3.30).

The result of particular interest for design is shown in Figure 3.30 and contains all the data at failure of the lining in terms of the ultimate thrust at failure (T_u) divided by the axial failure thrust (T_o) plotted against the linear eccentricity (e_ℓ) divided by the lining thickness (t). All the failure points fall in a rather narrow band and provide a means of obtaining the strength that includes the nonlinear behavior of the concrete by performing a linear analysis to obtain the eccentricity e_ℓ (by dividing the moment at the critical section by the

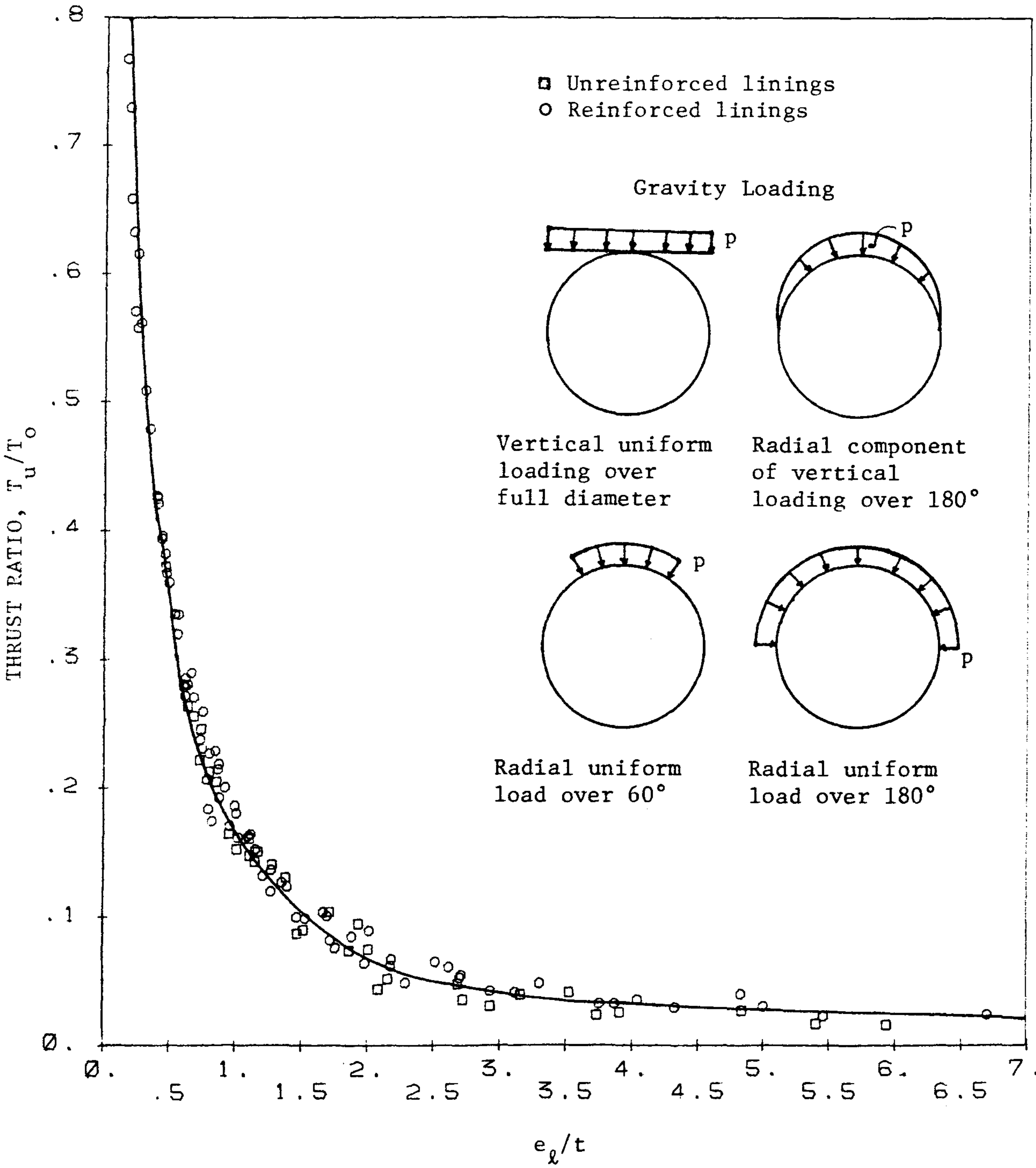


FIGURE 3.30 PREDICTION OF FAILURE OF LININGS BY THE NONLINEAR ANALYSIS AS A FUNCTION OF THE LINEAR ECCENTRICITY DIVIDED BY THICKNESS

corresponding thrust). The criteria used to determine failure was a concrete compressive strain of 0.004 or failure to converge in the analysis if this strain was not reached. Failure by this criteria always occurred after the interaction diagram was reached; for the largest eccentricity, failure occurred on the interaction diagram but for others the strain of 0.004 was reached shortly after the moment-thrust path turned inward after leaving the interaction diagram; normally there was some capacity of the lining remaining in the latter cases that was larger as the flexibility ratio increased. Also failure occurred at a higher thrust after leaving the interaction diagrams for the unreinforced lining than for the reinforced linings; that is, there was more reserve capacity remaining. The variation of compressive strain for a typical reinforced lining is shown in Figure 3.31 where the interaction diagram and moment-thrust paths are shown with the compressive strains indicated on the paths.

Figures 3.32 and 3.33 compare the various loading conditions in terms of p/f'_c and flexibility ratio (F). The pressure p for the excavation loading and overpressure loading are the full overburden pressure at the level of the tunnel rather than that acting on the lining (part of this pressure arches around the lining), while that for the gravity loadings is the pressure of a certain height of soil above the lining acting directly on the lining. Though these loading are not directly comparable it is convenient to place them on the same plot and instructive to compare them. It is clear that if full overburden pressure is used for the design for all the loading conditions, the gravity loadings would always govern, but this would be reasonable only for very shallow tunnels. For deeper tunnels the gravity loading would be limited to some depth of soil above the tunnel on the order of two diameters in the worst case and less in most cases as discussed in Chapter 1.

The excavation and overpressure loading cases were analyzed with joint elements between the linear medium and the lining to represent the shear stress on this surface. In the gravity loading cases where tangential springs are used to represent this shear stress, four

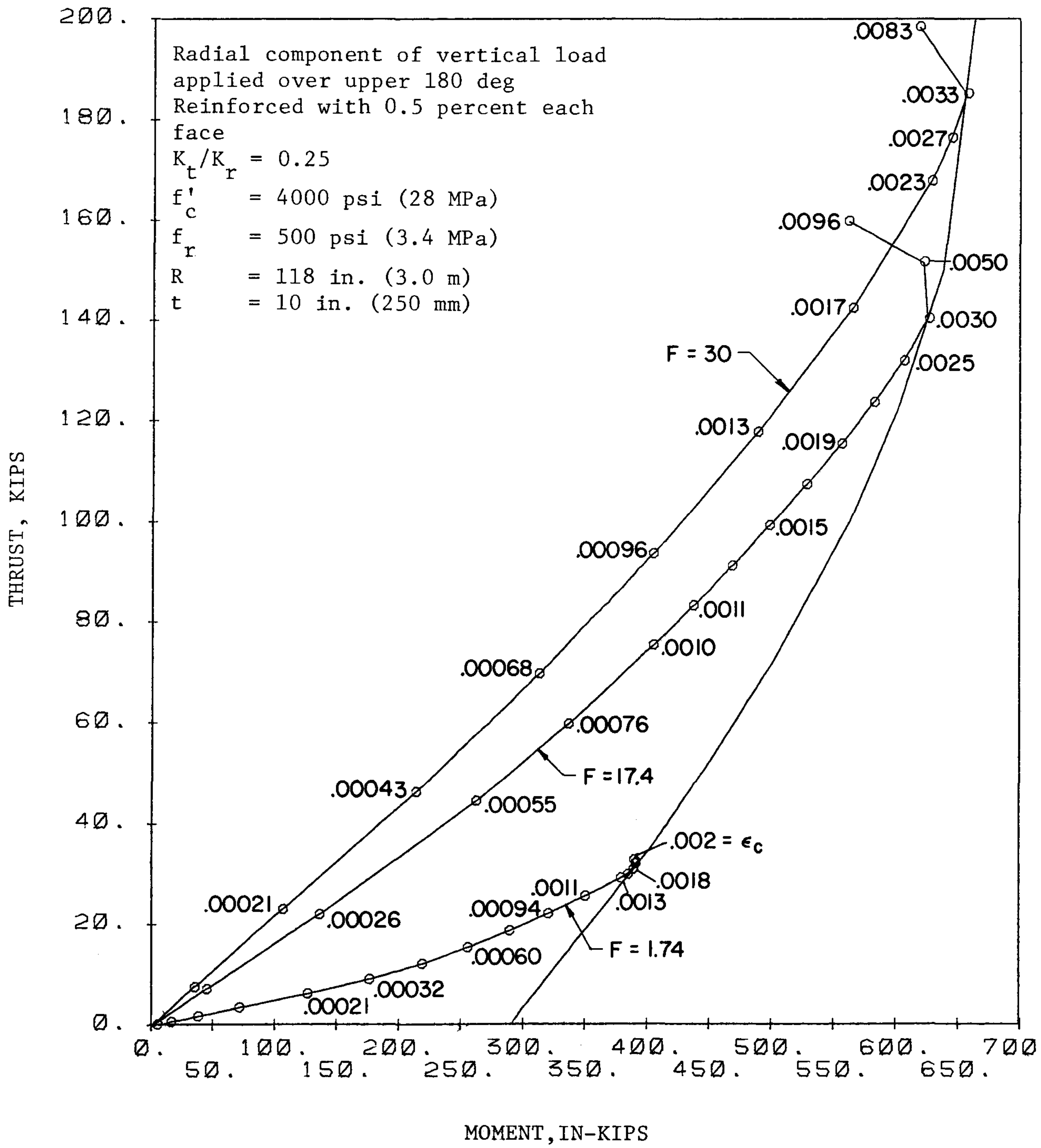


FIGURE 3.31 COMPRESSION STRAIN VARIATION ALONG THE MOMENT-THRUST PATH

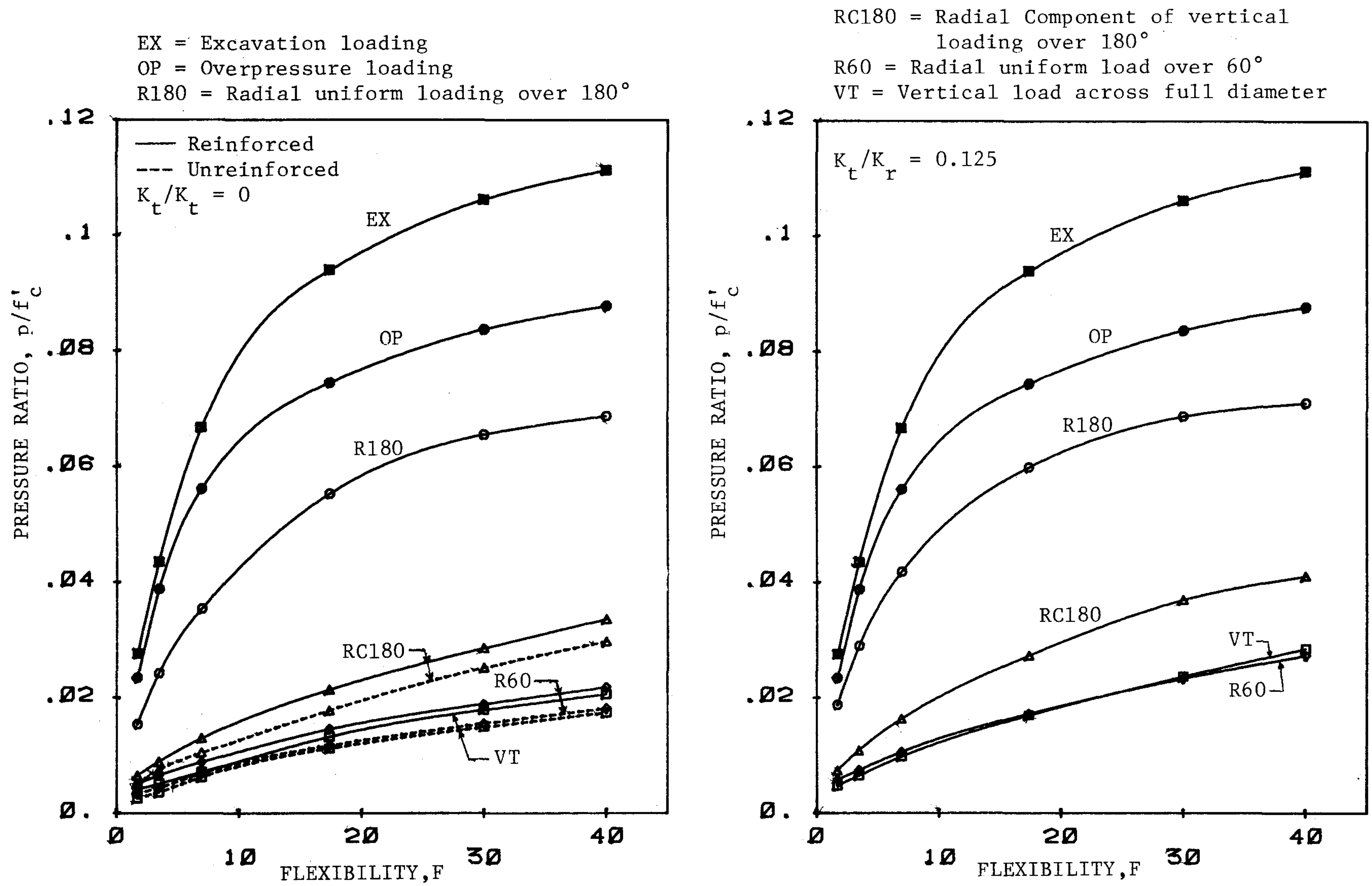


FIGURE 3.32 COMPARISON OF LOADING CONDITIONS FOR K_t/K_r OF 0 AND 0.125 FOR GRAVITY LOADING IN TERMS OF p/f'_c

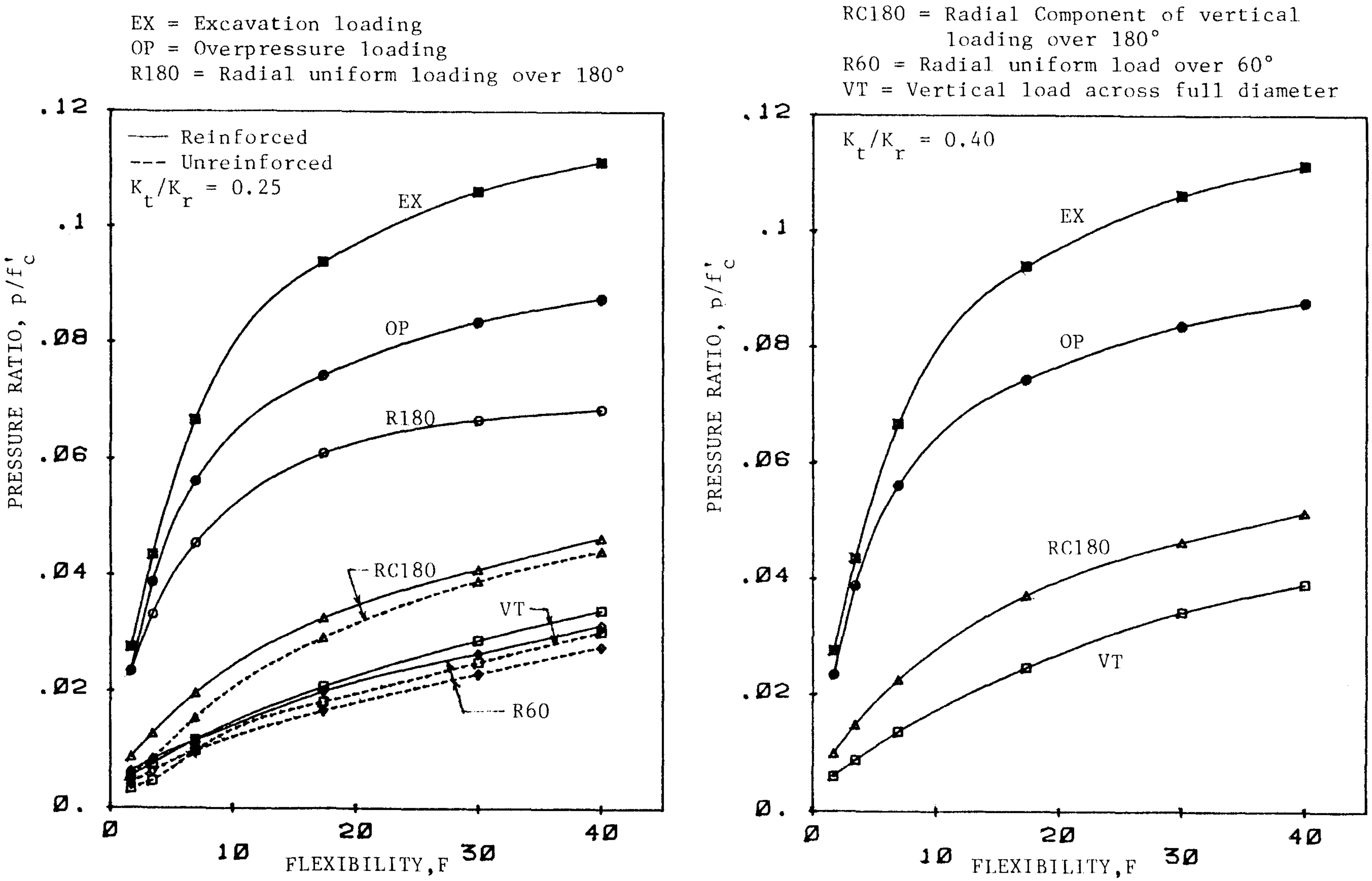


FIGURE 3.33 COMPARISON OF LOADING CONDITIONS FOR K_t/K_r OF 0.25 AND 0.40 FOR GRAVITY LOADING IN TERMS OF p/f'_c

different values were investigated and it is clear from Figures 3.32 and 3.33 that increasing the tangential spring stiffness relative to the radial stiffness increased the lining strength, and the increase is greater as the flexibility ratio increases because the absolute value of the spring stiffness is also increasing.

Figures 3.32 and 3.33 also show that reinforcement increases the pressure on a lining required to cause failure, because the reinforcement increases the thrust capacity, but this thrust capacity can easily be replaced by a very small increase in concrete strength or thickness. The curves of Figures 3.34 and 3.35 show the data in the previous figures except that the flexibility ratio is plotted against T_u/T_o defined earlier, and the difference between the reinforced and unreinforced cases is reduced. This shows that the reduction in thrust capacity by moment is essentially the same for a reinforced and unreinforced lining.

The results for the vertical, radial component of vertical and radial over 60 deg loadings are shown in Figure 3.36 in terms of p/f'_c vs F. Here it is clear what effect the tangential shear has. Above F = 20, for example, the shear spring stiffness of 40 percent of the radial values can increase the lining capacity by 50 to 100 percent. The effect of reinforcement is also shown, and its influence appears to be constant throughout the range of F except for values below about 10, where the capacity of the unreinforced lining appears to decrease more rapidly for the radial component of vertical.

Cracking in linings can be evaluated by studying the strains that occur at critical sections during loading; in Figures 3.37 and 3.38 contours of tensile strain at the tension face of the concrete are shown on the moment-thrust paths for the radial component of vertical loading, $K_t/K_r = 0.25$ and the reinforced and unreinforced linings. From these curves the thrust level can be seen relative to the ultimate thrust for a particular strain reached. Flexural cracking is generally considered to occur at about 0.00015 strain for a fairly rapid loading, but if the loading is slow it is reasonable to double this. Therefore,

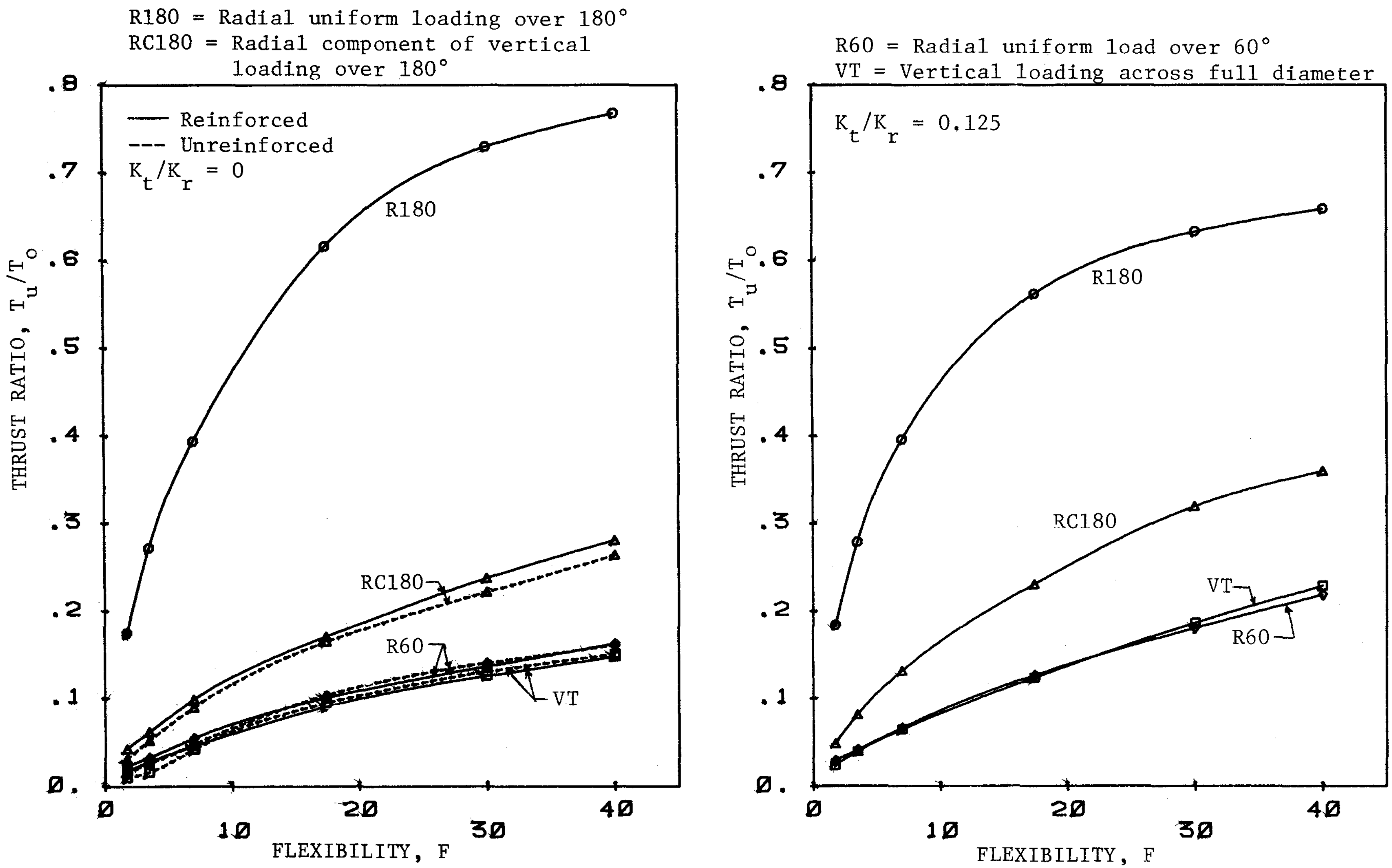


FIGURE 3.34 COMPARISON OF GRAVITY LOADING SHAPES FOR K_t/K_r OF 0 AND 0.125

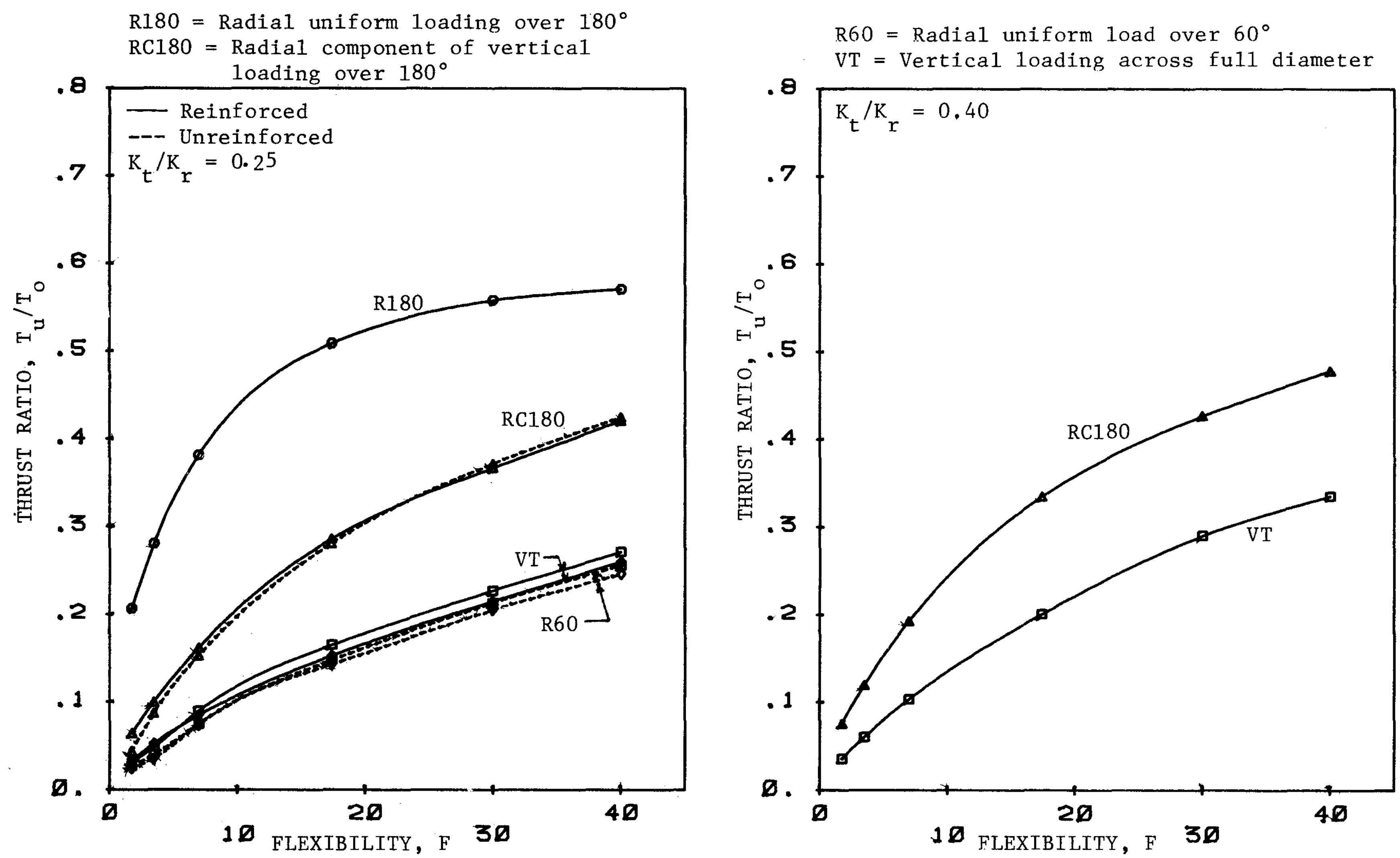


FIGURE 3.35 COMPARISON OF GRAVITY LOADING SHAPES FOR K_t/K_r OF 0.25 AND 0.40

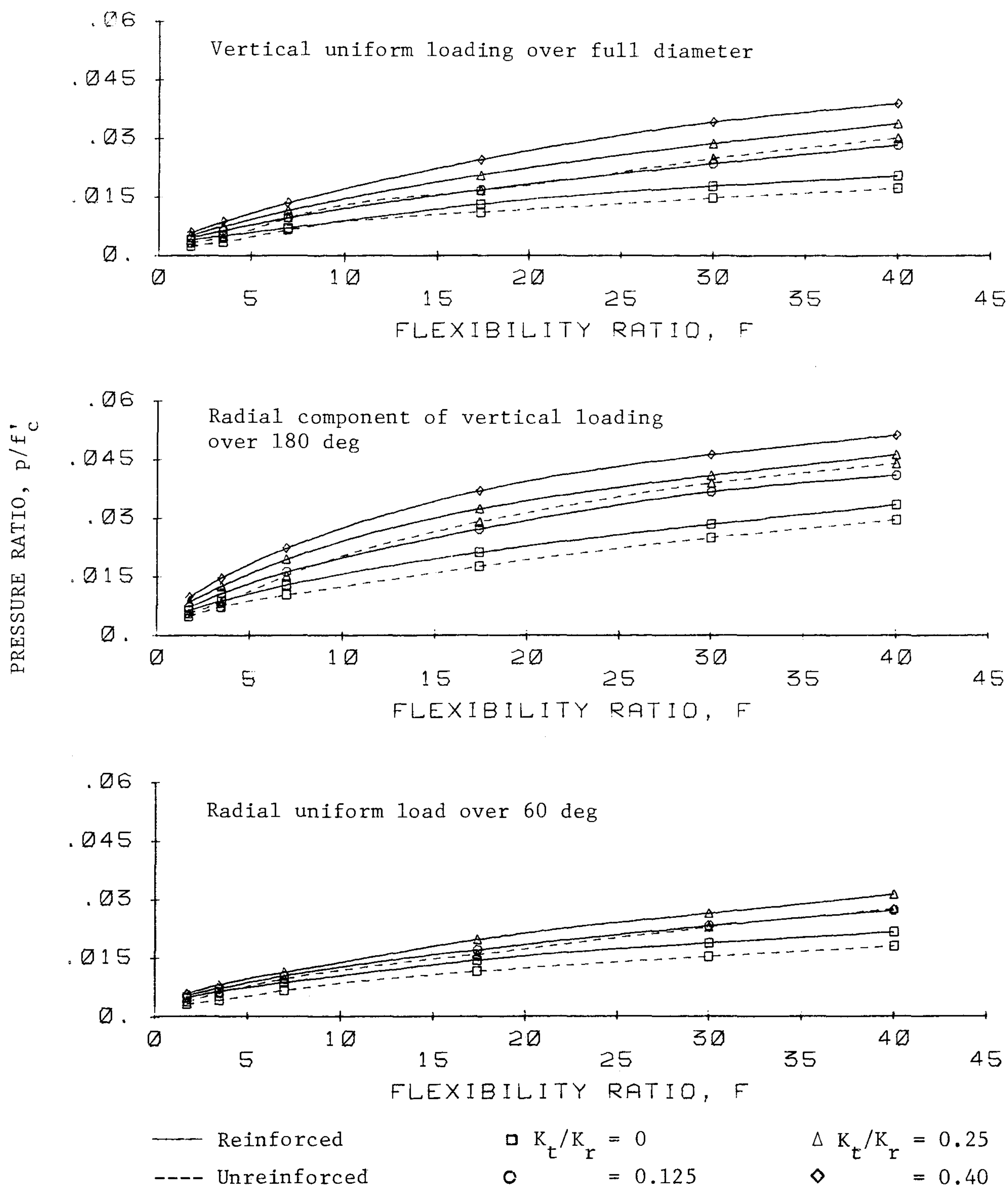


FIGURE 3.36 EFFECT OF K_t/K_r FOR THE VARIOUS GRAVITY LOADING SHAPES

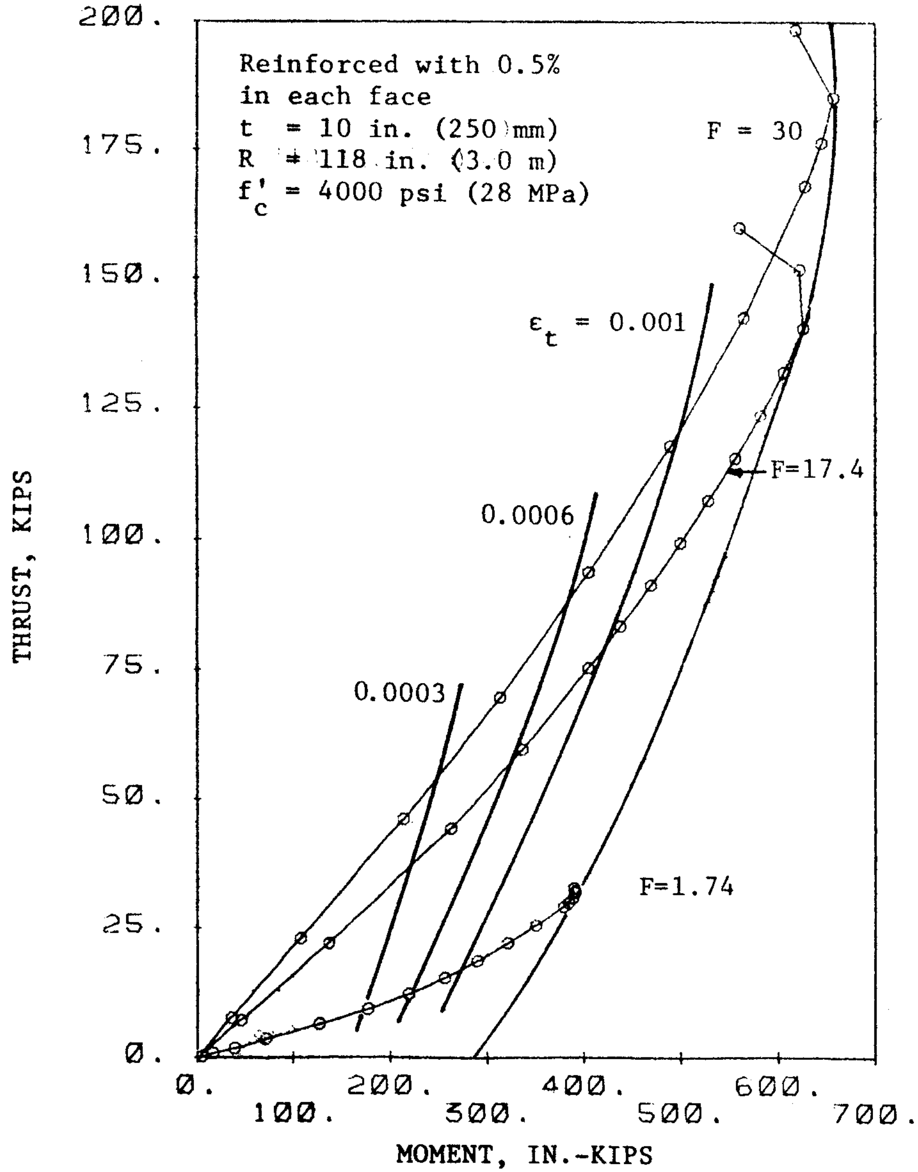


FIGURE 3.37 TENSION STRAINS ALONG THE M-T PATH FOR A REINFORCED LINING

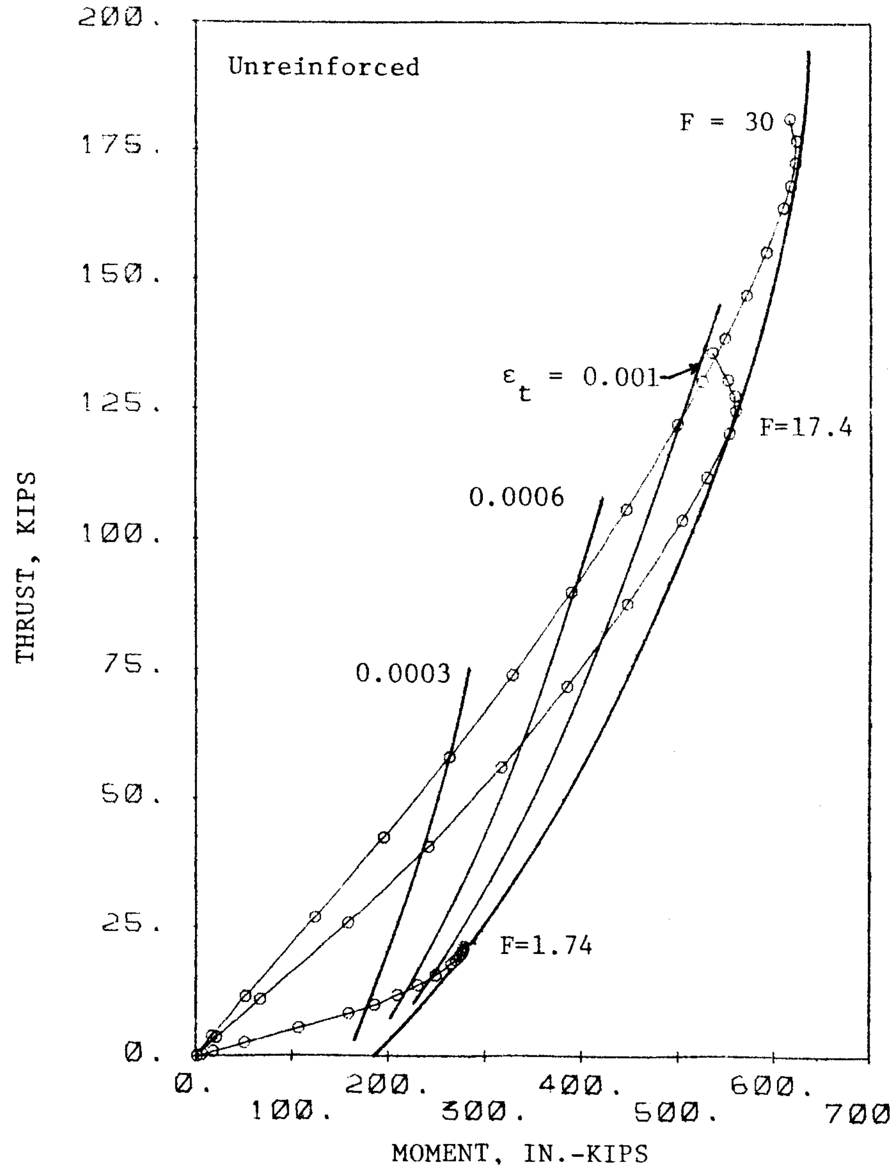


FIGURE 3.38 TENSION STRAINS ALONG THE M-T PATH FOR AN UNREINFORCED LINING

the 0.0003 contours in these figures may be considered to approximately indicate cracking in linings. In that case cracking occurs at about 25 to 30 percent of the ultimate thrust for the reinforced lining for the range of F shown; for the unreinforced lining the cracking thrust is about the same level relative to the ultimate for the two larger values of F, and is about 40 percent for the smallest F. The range remains the same (25 to 30 percent) for the reinforced linings for all loading conditions, and for the unreinforced lining it is between 29 and 37 percent for the larger values of F and 40 to 50 percent for the smallest.

After cracking the width of cracks can be estimated. For the reinforced lining a strain of 0.0006 at the critical section and crack spacing of 10 in. (254 mm) would correspond approximately to a crack width of 0.006 in. (0.15 mm), and the thrust level at which this would occur can be observed on Figure 3.37. The width would likely be larger for the unreinforced lining because the spacing would be larger.

Effect of Reinforcement: Three different reinforcement ratios (0, 0.5 and 1.0 percent) were used to investigate their effect on the lining behavior. The moment-thrust paths of the critical sections for failure are shown in Figure 3.39 for different flexibility ratios. It will be noted that the paths for different reinforcement ratios do not change in the linear range. Although they approach their respective envelopes showing higher ultimate thrusts for higher reinforcement ratios, the total ultimate load does not vary appreciably (Figure 3.40). The moment-thrust paths for the critical section for cracking, which is the crown, is plotted in Figure 3.41, which also includes the cracking envelopes according to cracking strains of 0.00015 and 0.00030. Since the cracking envelope remains essentially the same for all three reinforcement ratios, the cracking loads are also not affected by variation in reinforcement. This diagram also points out that cracking may be a problem for low flexibility ratios. The same trend has been noticed for excavation loading also. However, if the cracking strain is increased to 0.0003 as a result of creep and the slow application of load, cracking is greatly reduced. Only below a

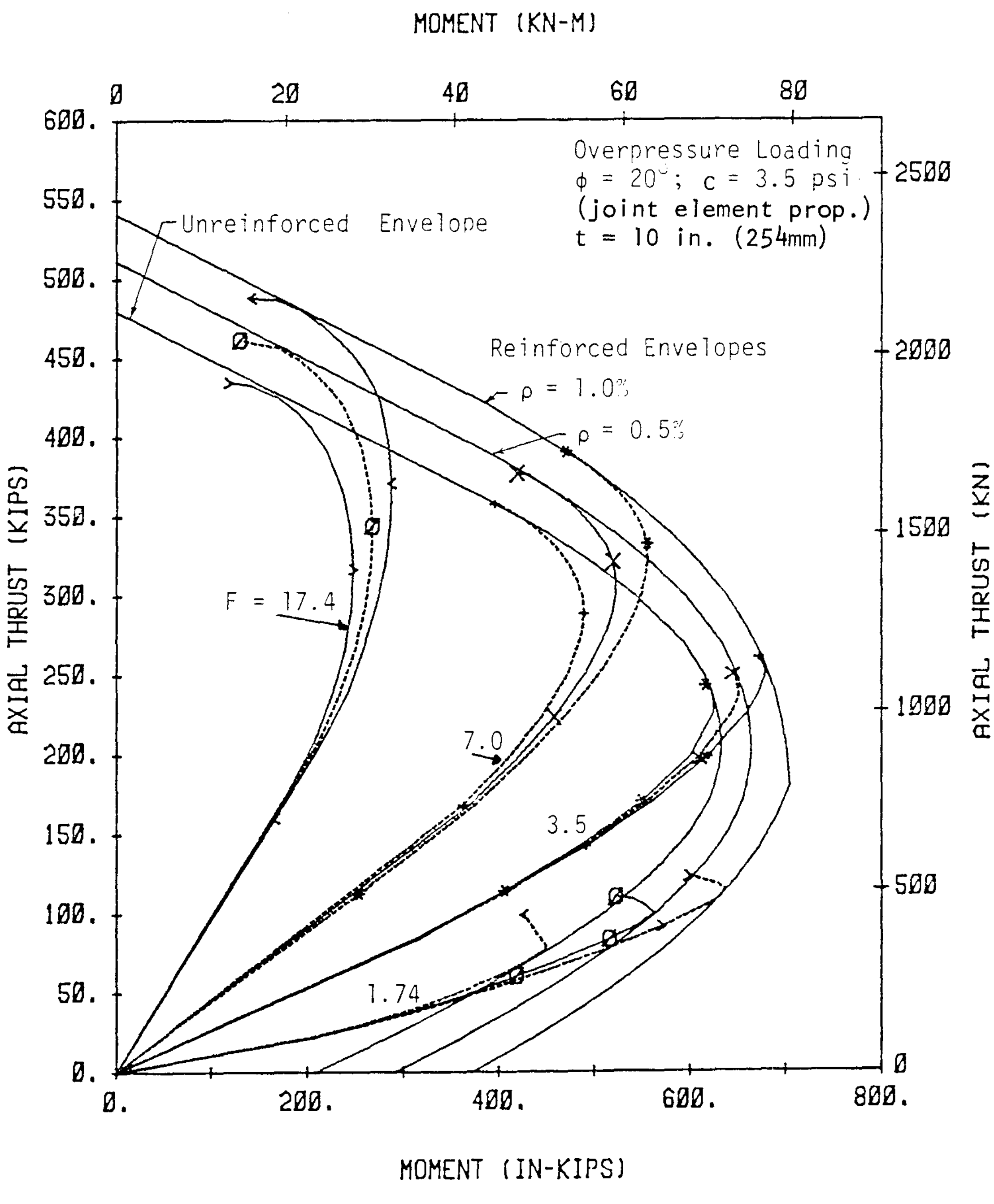


FIGURE 3.39 EFFECT OF REINFORCEMENT ON MOMENT-THRUST PATHS FOR OVERPRESSURE LOADING

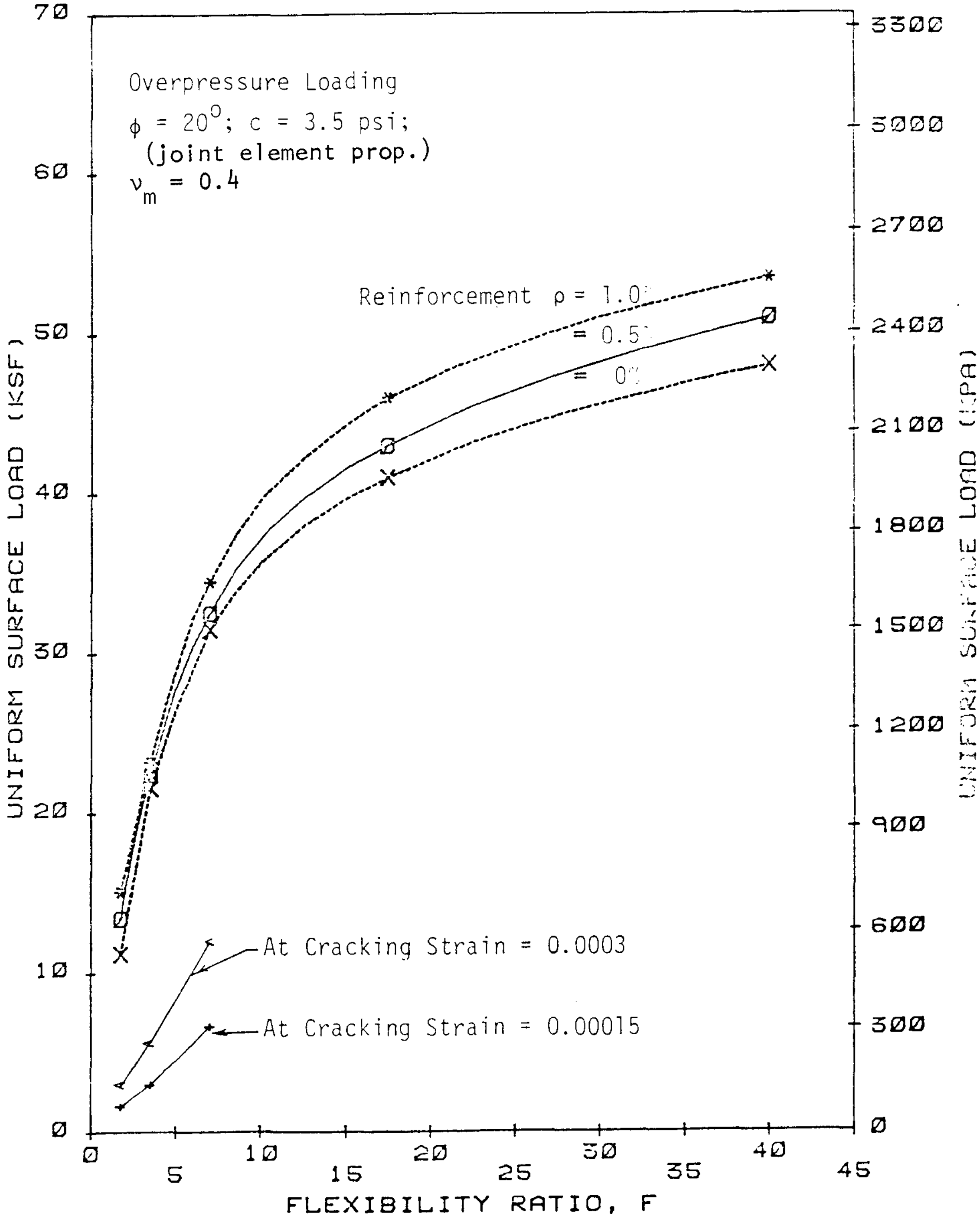


FIGURE 3.40 EFFECT OF REINFORCEMENT ON TOTAL LOAD AND CRACKING LOAD

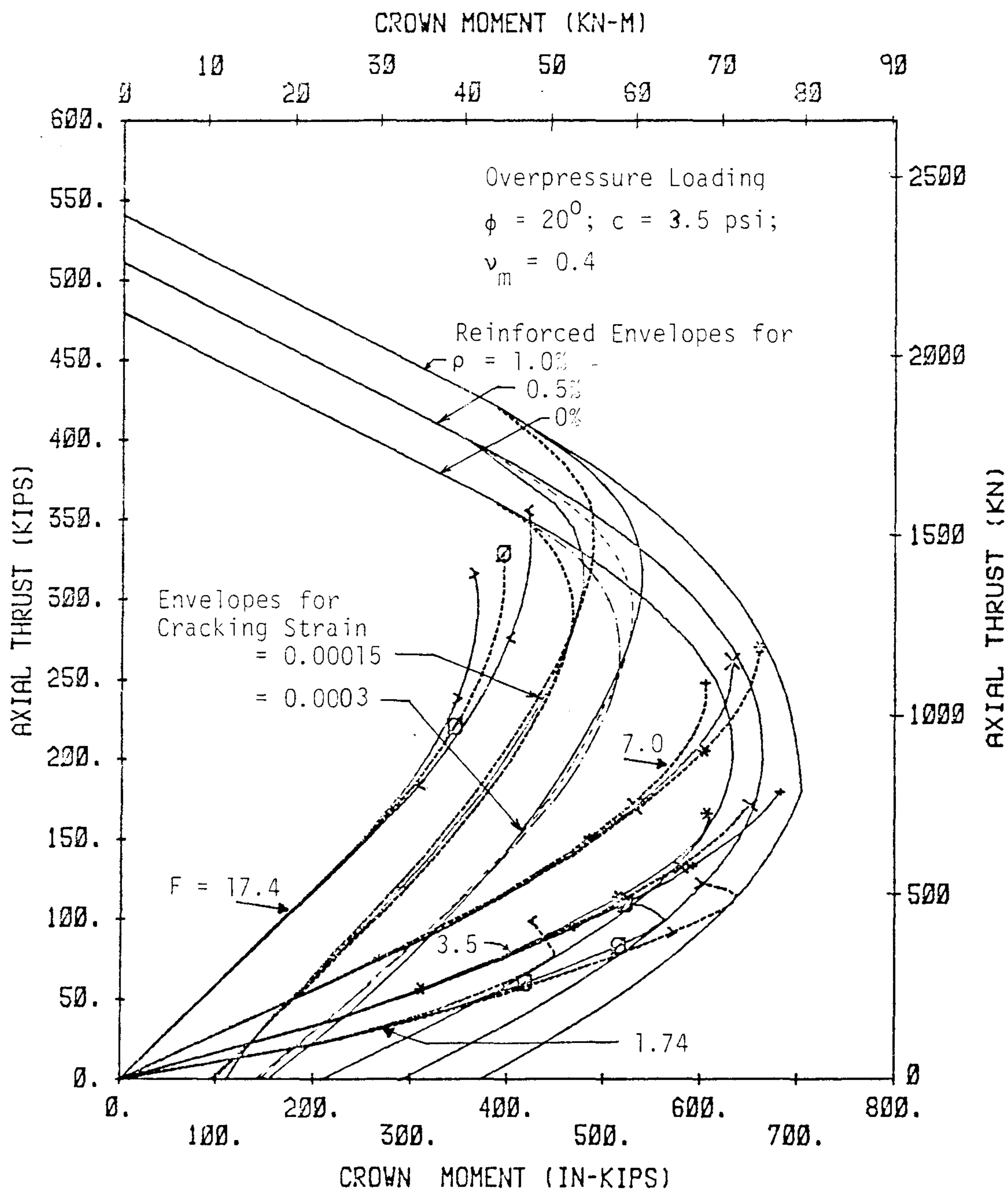


FIGURE 3.41 EFFECT OF REINFORCEMENT ON FIRST CRACKING FOR OVERPRESSURE LOADING

flexibility ratio of about 10 is flexural cracking likely to be a problem, and the relative values of cracking load and ultimate load are shown in Figure 3.40 in terms of uniform surface pressure. The effect of doubling the cracking criterion is shown, where the ultimate load shown is the actual ultimate based on the nonlinear analysis. Minor cracking due to flexure is not likely to create leakage because the compression zone of concrete remains an effective barrier to water passage. However, since reinforcement does help in reducing crack widths by distributing them, it may be advisable to use it in the amount required from the cracking point of view if it is calculated to occur and will create problems. An addition of a reasonable amount of reinforcement does not appreciably improve the load carrying capacity of the lining, however.

Effect of Joints in Segmented Linings: The presence of joints in segmented linings causes reduction in the stiffness of the lining, which in turn increases the flexibility ratio and thus reduces the moments in the li@G This reduction in the lining stiffness was obtained from the analysis for a set of typical dimensions by calculating an equivalent modulus of a monolithic lining (E_{eq}) that will give the same moment coefficient (M/pR^2) as given by a segmented lining of the same thickness. Using the closed form analytic solution described by Ranken, Ghaboussi and Hendron (1978) for linear analysis with excavation loading and full slip conditions, the moment coefficients for different values of elastic moduli of the lining (E_ℓ) and of medium (E_m) were obtained for a monolithic lining of 8 in. (200 mm) thickness and 19 ft 8 in. (6 m) diameter (Figure 3.42). Two types of segmented linings with eight and four segments per ring with a joint always at the crown were analyzed using the beam-continuum model. The joints were represented by very short unreinforced beam elements with concrete stress-strain properties without tension. The maximum moment coefficient obtained for the lining from the analysis of a segmented lining is entered in Figure 3.42 to obtain an equivalent modulus of elasticity of the lining (E_{eq}) that would give the same moment coefficient for a monolithic lining. The ratio of this modulus to the original modulus of elasticity of the segmented lining is plotted in

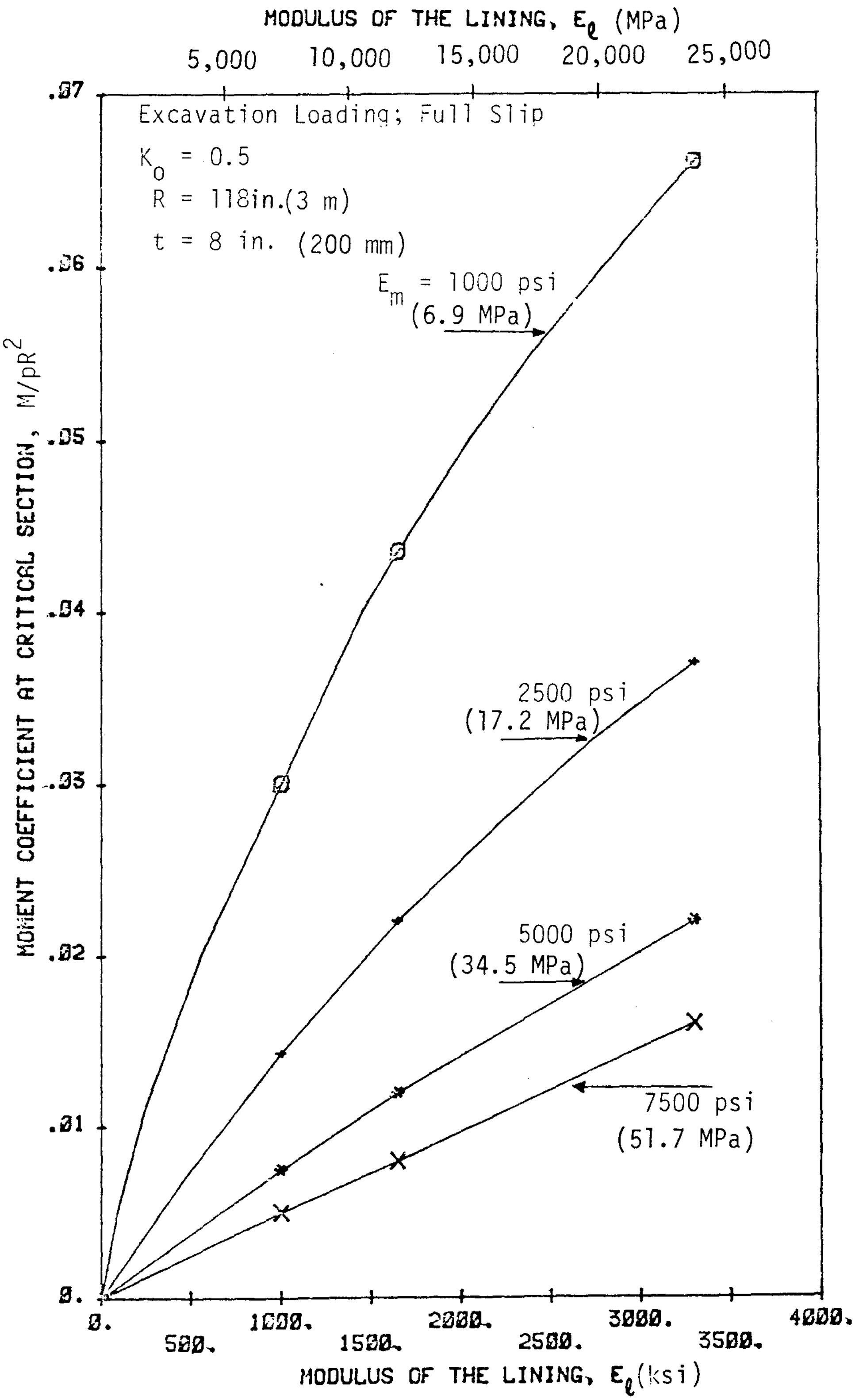


FIGURE 3.42 MOMENT COEFFICIENT VS MODULUS OF LINING FOR AN 8 IN. (200 mm) MONOLITHIC LINING IN DIFFERENT SOIL MEDIA

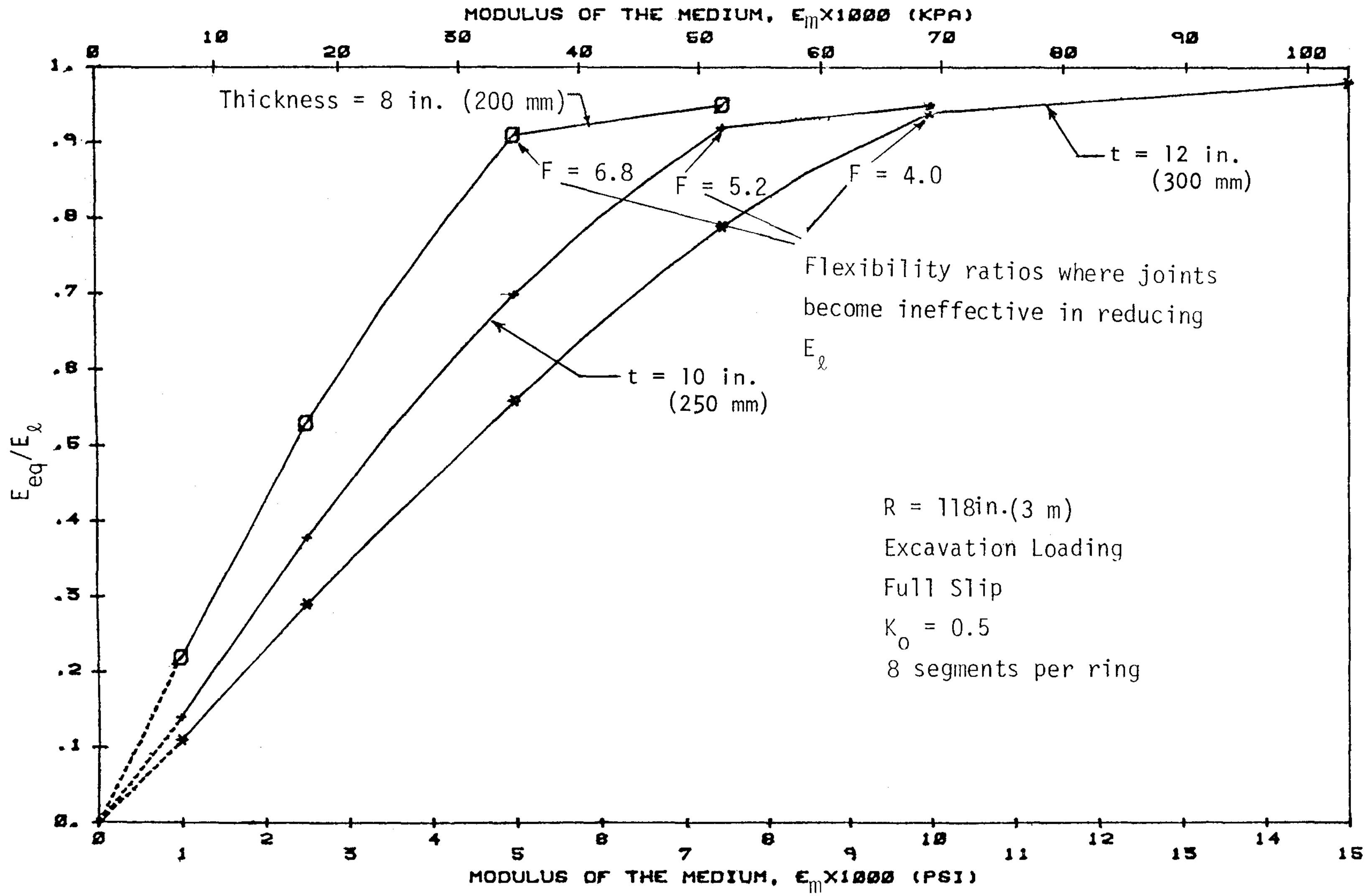


FIGURE 3.43 EQUIVALENT MODULUS OF ELASTICITY OF SEGMENTED LININGS

Figure 3.43 for different medium moduli and lining thicknesses. Almost identical curves were obtained for both four and eight segments per ring with slightly higher values of E_{eq}/E obtained in the lower range of medium modulus for four segments per ring. These results show that the stiffness of the segmented linings in the practical range of soft ground could be reduced by the effect of the joints to 30 to 95 percent of that of a monolithic lining with the same thickness. Once the modulus of the medium reaches a certain value, however, the joints become ineffective in reducing the stiffness and thus the moments in the lining (Figure 3.43), because of high thrust values that do not allow the joints to open. Thus, for certain combinations of lining thickness and medium modulus, segmented linings could be treated as monolithic from an analysis point of view. The number of joints per ring did not significantly influence the magnitude of moments in the lining for the particular joint orientations selected. It is, however, noted that the values of E_{eq}/E shown in Figure 3.43 should only be used as a guide in actual design problems, because of the numerous assumptions made in obtaining them (i.e., modeling of the joints, excavation loading, K_o value, specific radius of the opening, and specific joint orientation). Nevertheless, they provide an indication of the effect of joints, in conjunction with other parameters, on the lining stiffness and the order of magnitude of the reduction to be expected.

3.2.3 Parameter Study for Circular Linings in Rock

The beam-spring model used for the study of arches in rock and described in Section 3.2.1 was modified and used to examine the effects of various parameters when a loosening rock load is applied directly to a circular lining. The model used is shown in Figure 3.44 and a uniform loosening load was applied across the lining.

Parameters studied were the relative flexural stiffness of the medium to the lining described by the flexibility ratio F, relative stiffness of the tangential and radial springs K_t/K_r, radius to thickness ratio of the lining R/t, and the lining reinforcement. A

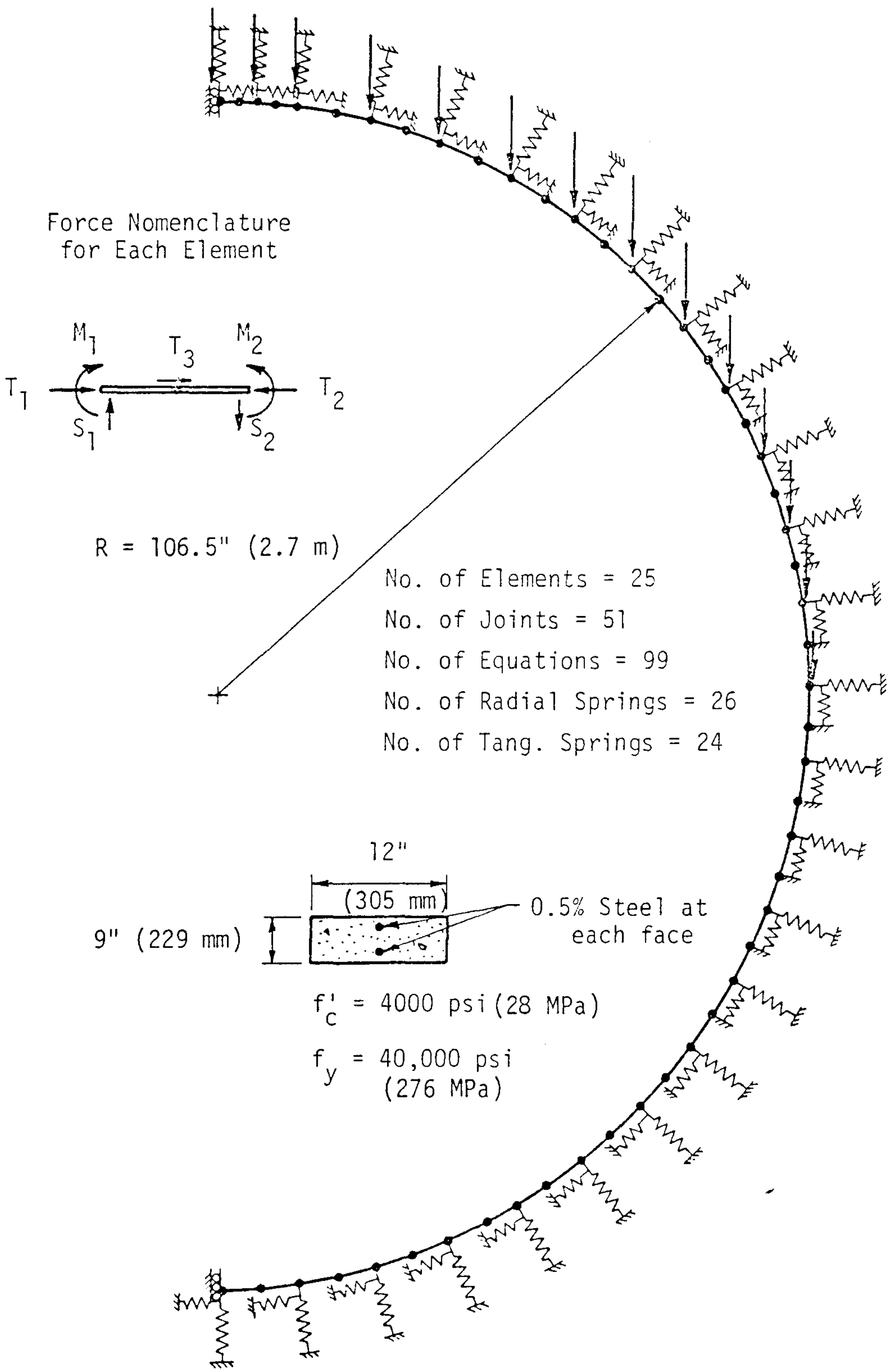


FIGURE 3.44 FINITE ELEMENT MODEL OF CIRCULAR LINING FOR PARAMETRIC STUDIES IN ROCK FOR LOOSENING LOAD

particular typical lining that was 9 in. (230 mm) thick with a radius of 106.5 in. (2.70 m) was used for most of the study, and then the thickness and reinforcement of the lining were changed to study the effects of R/t and reinforcement. The reinforced concrete section contained concrete with compressive strength of 4000 psi (28 MPa) and 0.5 percent deformed bar reinforcement in each face with a yield strength of 40 ksi (276 MPa). The concrete strength was changed to 4310 psi (30 MPa) for the unreinforced section so the axial thrust capacity would be the same as for the reinforced one in order to examine the effect of reinforcement on other aspects of behavior such as moment redistribution and ductility. The beam element used to represent the lining can have nonlinear behavior that depends on the material properties. The Hognestad equation was used for the compressive stress-strain curve for the concrete and the reinforcement stress-strain curve was elastoplastic.

Relative stiffness of the medium to the lining in flexure was determined by the flexibility ratio as presented by Peck, Hendron and Mohraz (1972), and the relationship between the radial spring stiffnesses and the elastic modulus of the medium given by Dixon (1971) was used. The formulas for these relationships are given in Section 4.3.4 of this report. The tangential spring stiffness was a variable.

The most important parameters affecting lining strength are the medium stiffness and tangential to radial spring stiffness ratio; the effects of these parameters on the lining strength in terms of the thrust ratio T_u/T_o is shown in Figure 3.45. If the lining has a 10 in. (250 mm) thickness and 19 ft (5.8 m) diameter, and is in a very soft rock with a Young's modulus of 100,000 psi (690 MPa) the F would be on the order of 70, provided the lining is uncracked. This is the range shown by the curves that have a steep slope so the T_u/T_o ratio can become low and is sensitive to the medium modulus selected. In this range care must be exercised in selecting the loading and the medium properties. If the lining cracks, its stiffness is immediately reduced relative to that of the medium, and if the cracked moment of inertia is one-half the uncracked value for example, the value of F is doubled and

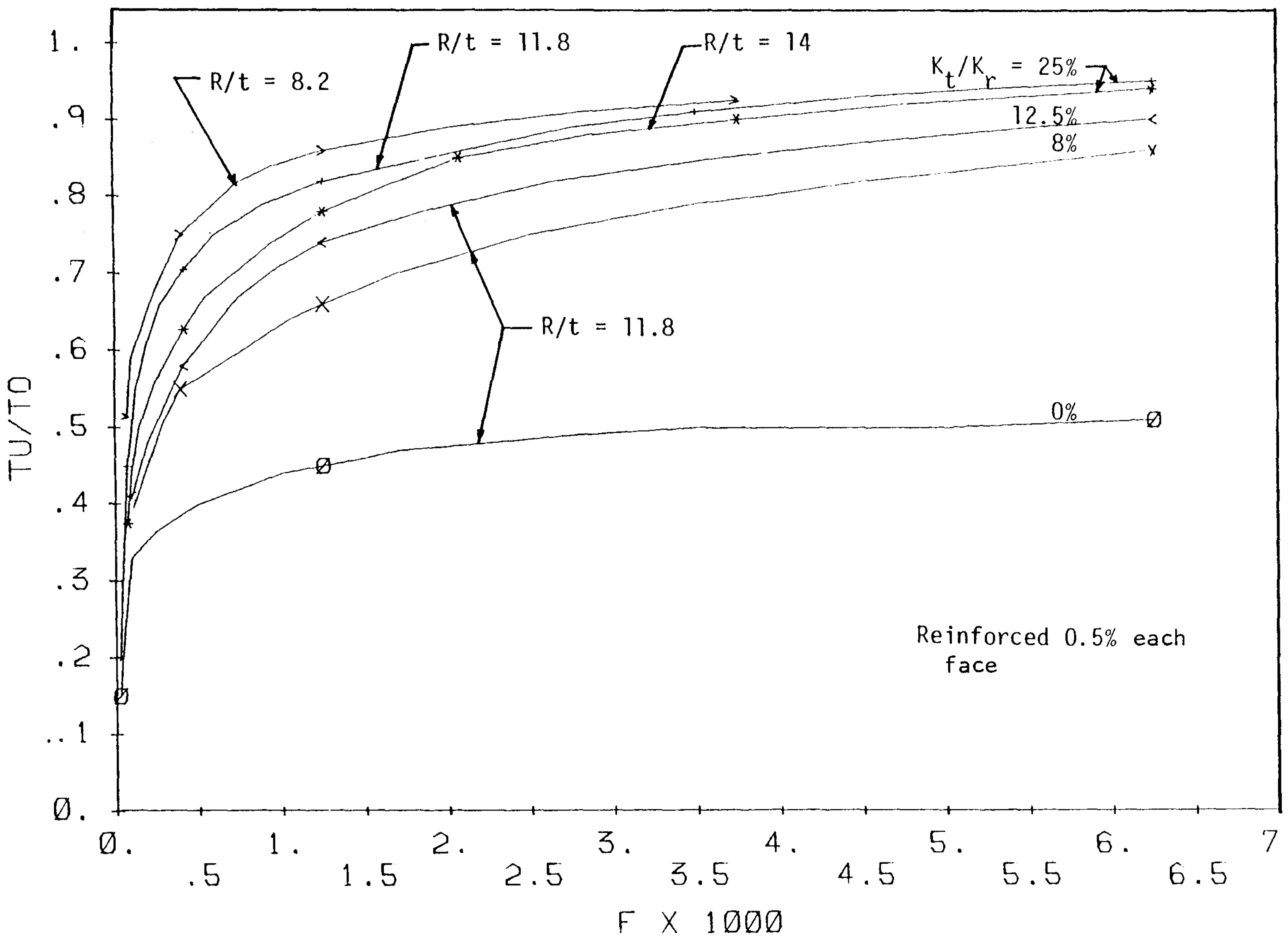


FIGURE 3.45 LINING CAPACITY AS A FUNCTION OF FLEXIBILITY RATIO

shifts to the right on the curves increasing the strength of the lining appreciably. When F is larger than 300, which corresponds to a deformation modulus of the medium of about 500,000 psi (3450 MPa) for the particular lining considered, the curves become rather flat, indicating a greatly reduced sensitivity to the medium modulus. In all cases analyzed in Figure 3.45, a plastic hinge occurred first at the crown and failure of the lining was precipitated by conditions at this location, though hinges started to develop at other location as well prior to collapse. In Figure 3.46 the vertical axis of Figure 3.45 is changed to show the actual uniform pressure on the lining at failure, where the definition of failure has been selected at a strain of 0.004 in the concrete. This is a conservative estimate of failure, as more load could be resisted beyond this point in most cases. The shape of the curves is very similar to that in Figure 3.45.

The effect of tangential spring stiffness is more easily shown on Figure 3.47 where the T_u/T_o ratio is plotted against the spring stiffness ratio for various values of F. These curves show that there is very little change in strength for all values of F when K_t/K_r is larger than about 0.12. Most designers agree that the effective K_t/K_r in the actual tunnel in rock is at least this large, so the analysis is not greatly sensitive to the value selected. The range normally used is from 0.10 to 0.50. If the value selected is smaller than 0.12 because the opening walls are smooth or a large amount of wood blocking remains between the final lining and the rock, then the effect of the value selected depends on the flexibility ratio; for low values of F the sensitivity remains small, and becomes larger as F increases.

The lining-medium system is not defined completely by the flexibility ratio, as shown in Figure 3.45 by the three curves with different radius to thickness ratios R/t with K_t/K_r of 0.25. The separation of the curves shows the effect of the R/t ratio. The reason for the variation with R/t is the effect of thickness on the nonlinear behavior of the lining, where the thicker lining has a larger T_u/T_o ratio at the same F. This variation of the strength of the

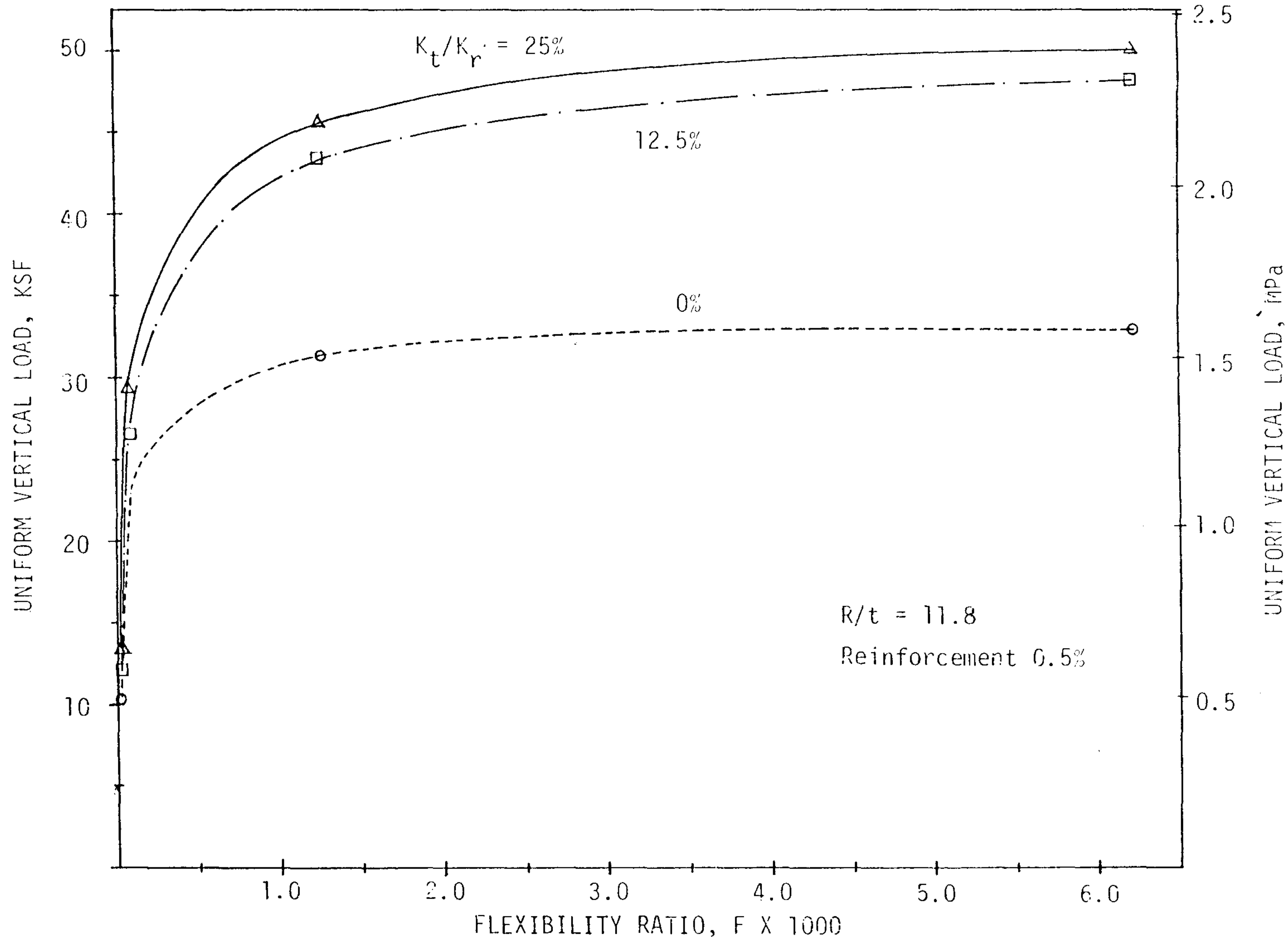


FIGURE 3.46 TOTAL LOAD VS FLEXIBILITY RATIO

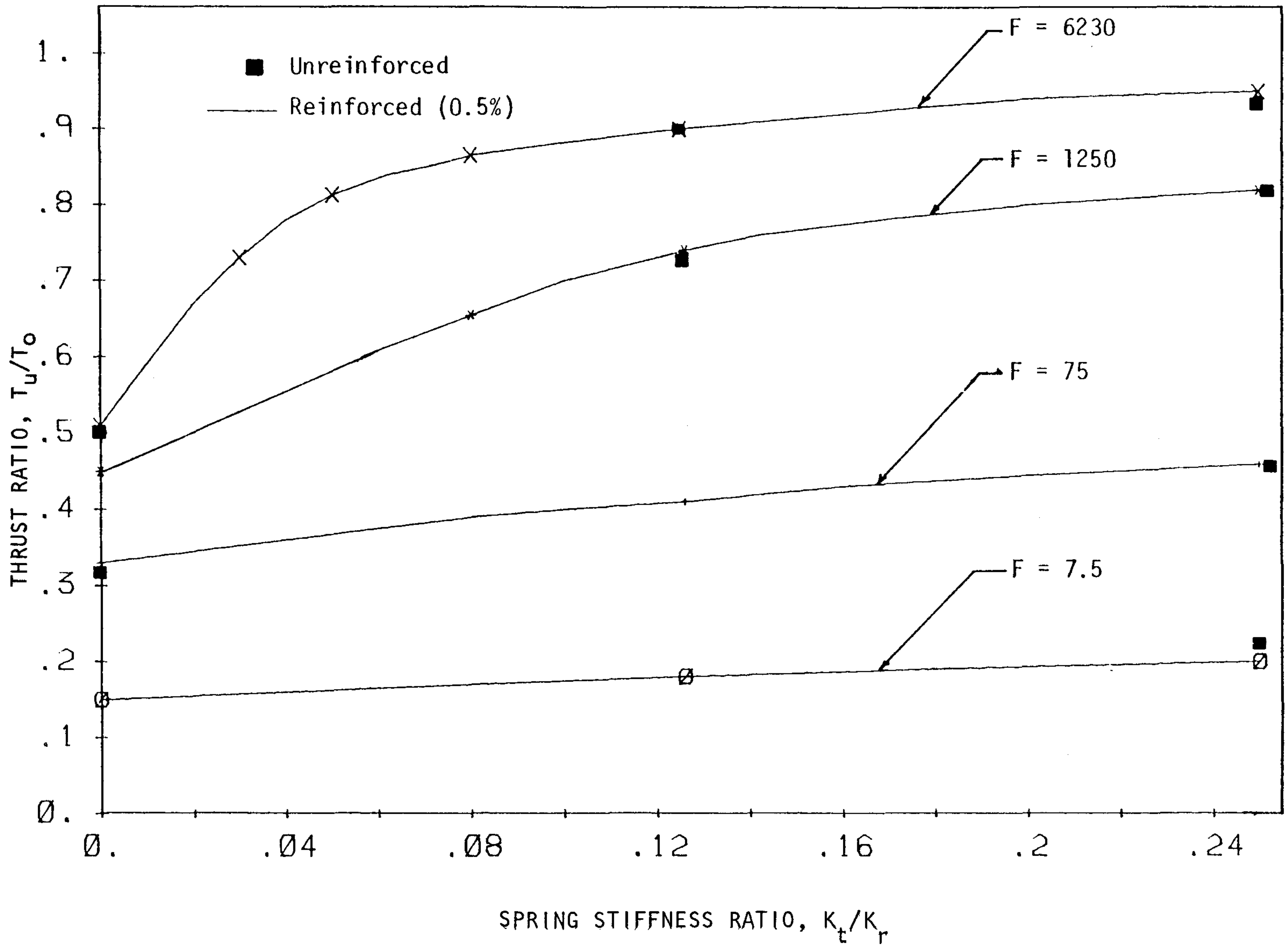


FIGURE 3.47 LINING CAPACITY AS A FUNCTION OF TANGENTIAL TO RADIAL STIFFNESS RATIO (K_t/K_r) FOR R/t = 11.8

lining as depicted by the thrust ratio is negligible for all practical purposes when the flexibility ratio is higher than 1250. However, for lower values of flexibility ratio, the radius to thickness ratio (R/t) can make appreciable difference as shown in Figure 3.45. If the radius and flexibility ratio are kept constant in a linear analysis, variation of the R/t ratio by changing the thickness of the lining does not affect the thrust ratio when the moment-thrust paths reach the envelope above the balance point; however, near the balance point the thrust ratio can become smaller for thicker linings while below the balance point it increases with the lining thickness. If a nonlinear analysis is performed, the effective flexibility ratio due to cracking and nonlinear stress-strain properties of the lining, will be increased more drastically for the thicker than for a thinner lining, so the thrust ratio is increased more for the thicker lining. Thus a thicker lining for the same radius and initial flexibility ratio, but lower R/t ratio, gives a higher thrust ratio.

The moment-thrust paths combined with the failure envelope for the critical sections in Figure 3.48 show how the flexibility ratio affects strength. When the flexibility ratio is small and therefore the deformation and moments are large, the moment-thrust paths reach the failure envelope below the balance point. The concrete and steel in the section are fairly ductile so the moment-thrust path follows the failure envelope until the concrete starts to crush on the compression side of the section. When crushing occurs, the internal thrust resultant shifts inward toward the center of the section, reducing the lever arm and therefore the moment, but the thrust can continue to increase as more concrete toward the tension side starts to resist compression. The reduction in moment causes the moment-thrust path to turn inward toward the thrust axis as shown for F = 7.5 and 75. The maximum thrust and thus the peak load is obtained when the rotational capacity of the critical section is finally reached. A value of F = 70 is about the lower limit for linings in rock and for this value the path normally approaches the interaction diagram near the balance point.

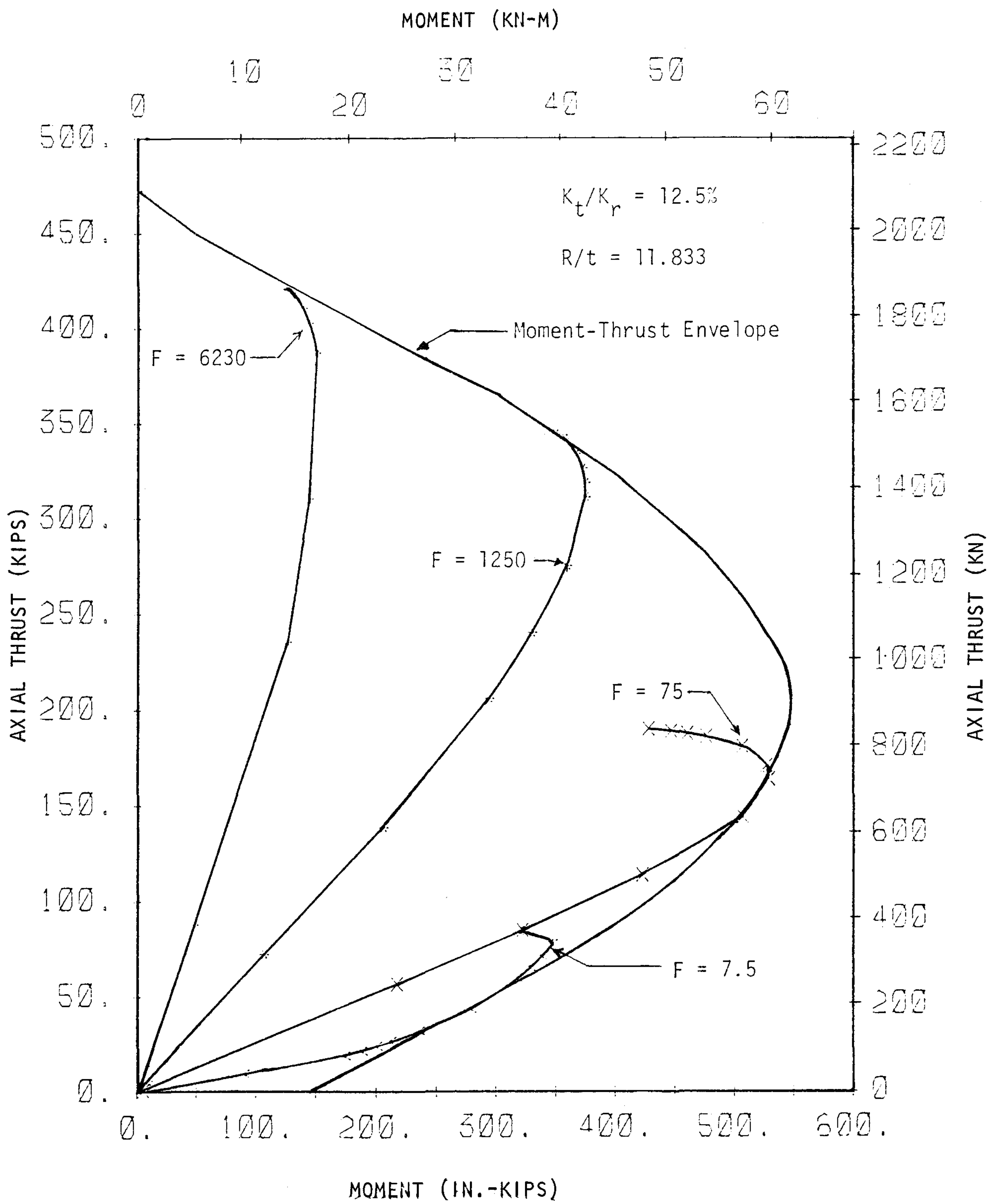


FIGURE 3.48 EFFECT OF F ON MOMENT-THRUST PATHS AT CROWN

When the flexibility ratio is large, as for F = 1250 and 6230 in Figure 3.48, the lining deformation and therefore the moment at the critical sections are smaller, so the moment-thrust path reaches the failure envelope above the balance point where the rotational capacity of the section is much smaller due to the large thrust. In this case the limiting rotation of the section is reached when the failure envelope is reached and there is no following of the envelope, but the thrust and therefore the load on the lining is considerably larger.

Since the ultimate load is nearly proportional to the thrust capacity, and the thrust capacity depends on where the moment-thrust path reaches the failure envelope, Figure 3.48 also shows how the flexibility ratio affects strength of the lining. The moment-thrust paths start as if the problem solution is linear, before the nonlinear effects begin. If the initial path is projected linearly to the intersection with the failure envelope the linear prediction of failure thrust would be obtained. Therefore, this figure can be used to visualize the difference between the linear and nonlinear prediction of failure, and it can be seen that above the balance point a linear prediction of the failure thrust is much more accurate than below the balance point. It also shows that the relationship between the stiffness of the medium and intersection of the moment-thrust path with the envelope is highly nonlinear.

The moment distribution around the lining at the maximum load is shown in Figure 3.49 for several values of F. Peak moment of opposite sign occur at the crown and at about 45 deg from the crown. For F = 7.5 and 75 the moment path reaches the failure envelope below the balance point (Figure 3.48) where an increase in thrust increases the moment capacity so the moment is larger for the larger F; however, for the two larger values of F an increase in thrust reduced the moment capacity above the balance point, so the moment is smaller for the larger F. Although peak moment at about 45 deg from the crown may become slightly larger than that at the crown, the thrust at this section is higher, so the crown still governs the design. In the lower

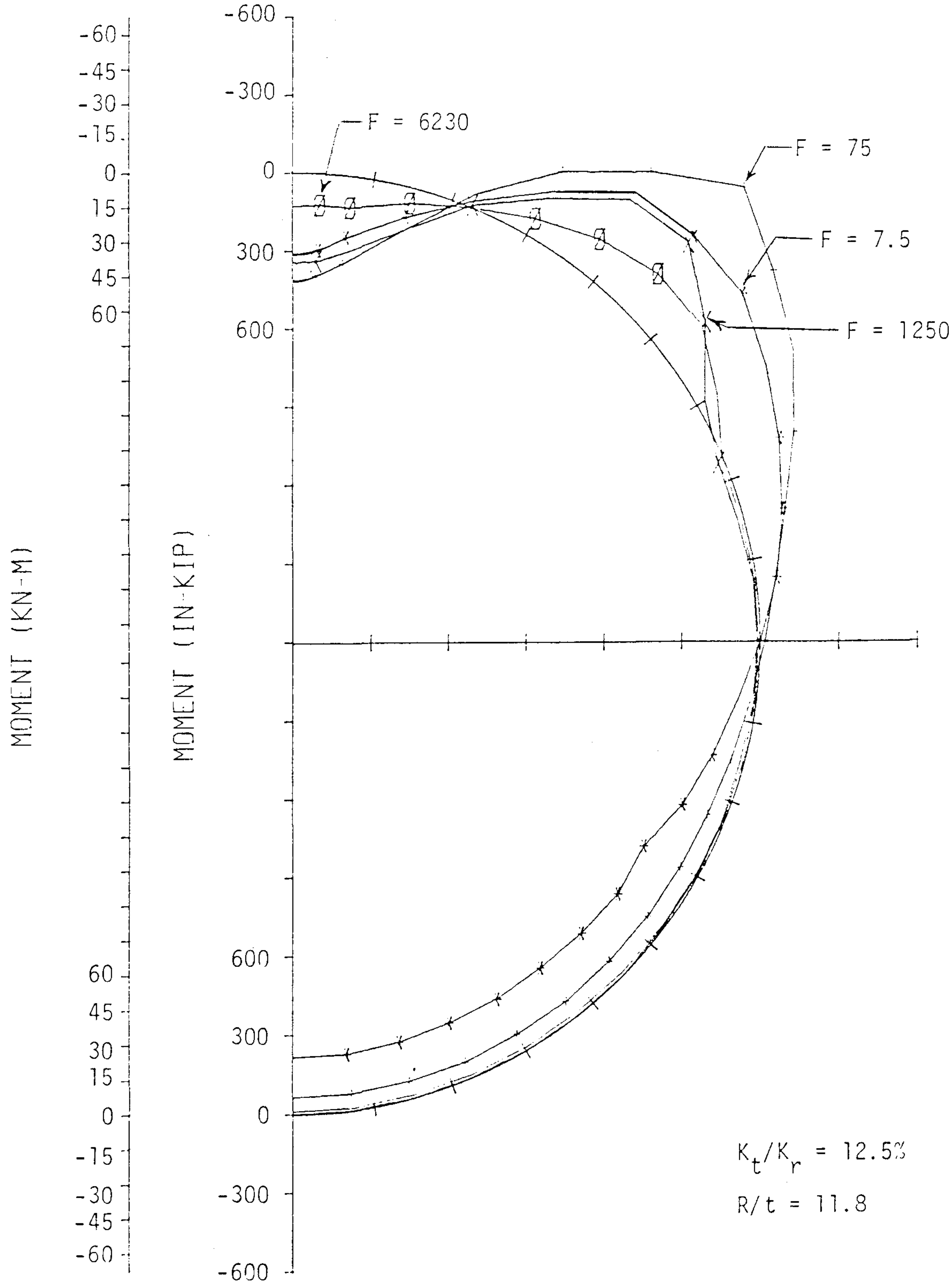


FIGURE 3.49 EFFECT OF F ON MOMENT DISTRIBUTION AT MAXIMUM LOAD

portion of the lining the moments are fairly small and are not likely to govern the design.

Internal shear distribution induced in the lining is shown in Figure 3.50 for the maximum loads. There are regions of peak shear near 20 deg and from 45 to 60 deg from the crown. The shear is larger at 45 to 60 deg for the high flexibility ratio and at 20 deg for the low values. The section closer to the crown are likely to be more critical, however, because the direction of diagonal tension at this section leads to movement into the tunnel of the loaded region near the crown.

Most of the solutions obtained for these comparisons were obtained for a lining with 0.5 percent reinforcement in each face, and then several problems were worked for a lining without reinforcement, but with f_c increased to compensate for the loss of axial thrust by removal of the reinforcement. The T_u/T_o ratios are compared for the reinforced and unreinforced linings in Figure 3.47 where there is essentially no difference. Reinforcement had a negligible effect on the stiffness of the lining for the larger values of F and increased the stiffness only slightly near failure for the low values of F. Moment-thrust paths are compared with and without reinforcement and with the failure envelopes in Figure 3.51. The envelopes are the same in the high thrust range but different below about 300 kips (1330 kN) where the moment capacity is increased by the reinforcement. There is little difference in the moment-thrust paths except for the lowest F; in this case the paths start to separate at a thrust of about 90 kips (400 kN) and each path approaches its respective envelope; the maximum moment that the reinforced section can resist is larger, but both paths turn back toward the thrust axis and reach essentially the same peak thrust so the load on the lining is essentially the same in the two cases. Therefore, in the range of flexibility ratio appropriate for linings in rock, reinforcement in the lining adds little strength or ductility in circular linings. Reinforcement is needed only in some cases for serviceability considerations, provided the design loads actually occur, because at service loads the thrust is large enough to prevent appreciable tensile stresses.

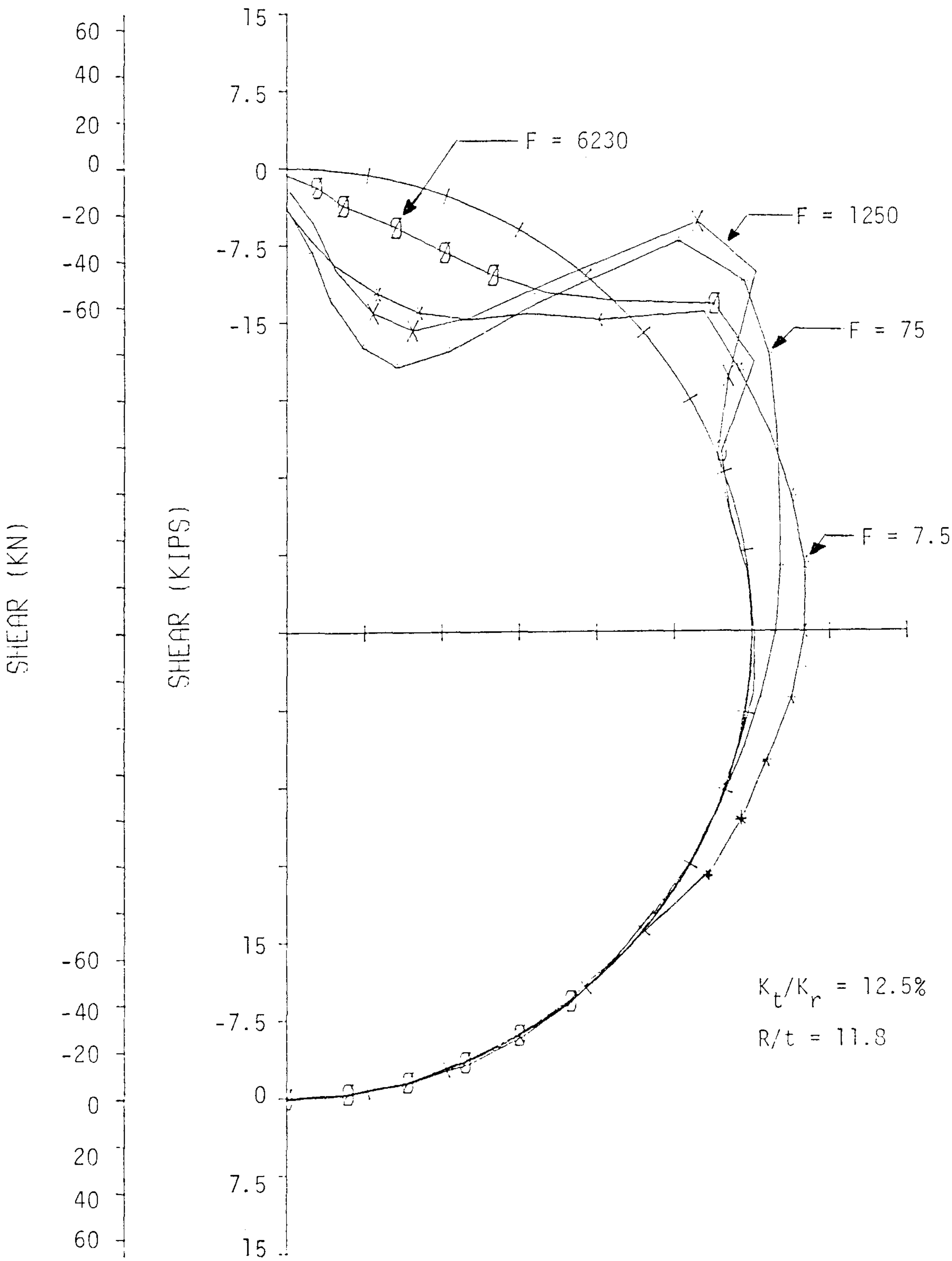


FIGURE 3.50 EFFECT OF F ON SHEAR AT MAXIMUM LOAD

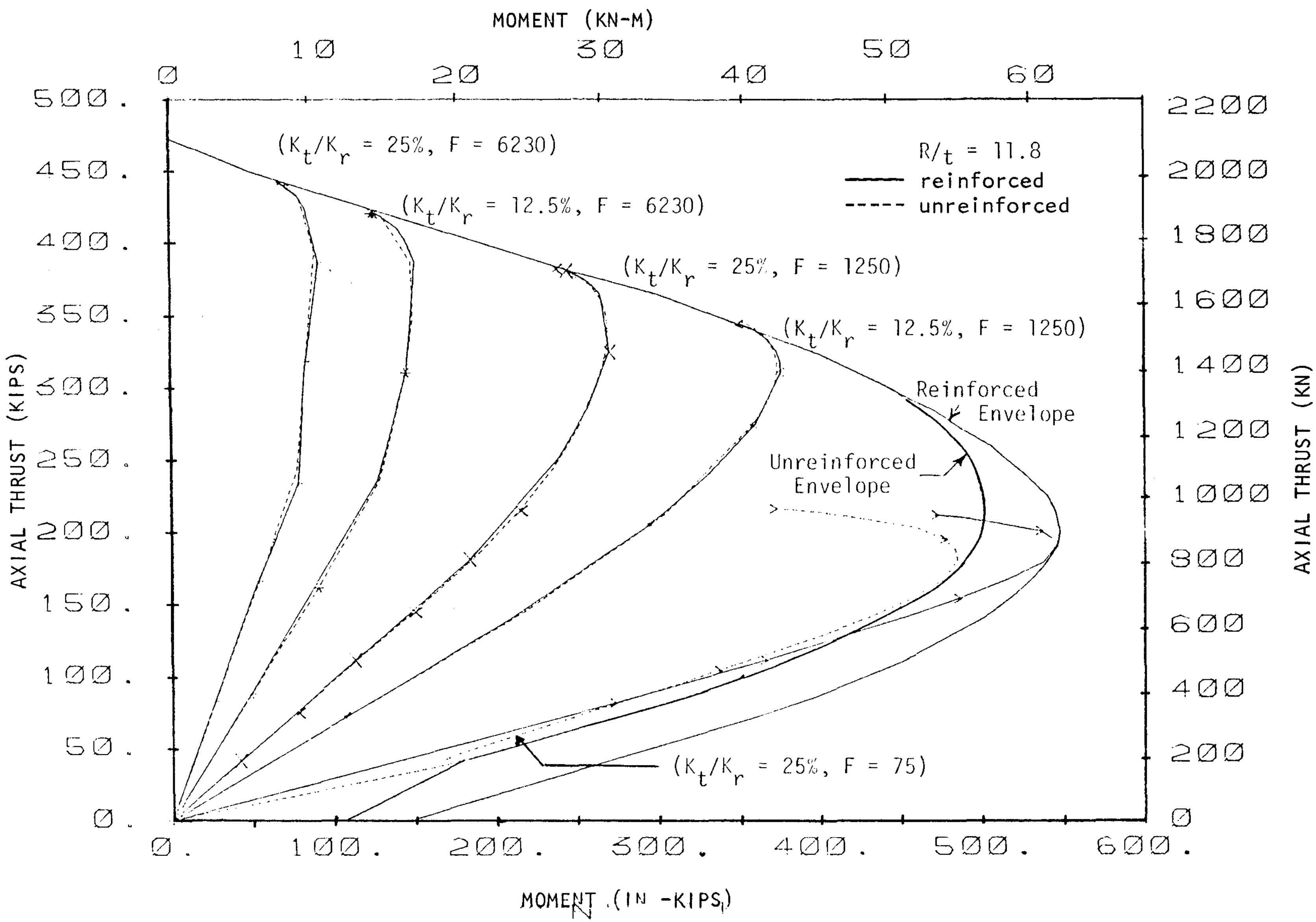


FIGURE 3.51 EFFECT OF REINFORCEMENT ON MOMENT-THRUST PATHS

CHAPTER 4

DESIGN RECOMMENDATIONS

4.1 SCOPE

It is well known that concrete and masonry linings in tunnels have the capability of supporting substantial loads, even though unreinforced, and even though the ground loads and distortions may be sufficient to cause tension cracks to form and open. As a tunnel lining in continuous contact with the soil deforms and approaches its bending capacity, it will continue to build passive reaction against the soil or rock that will prevent the unrestricted deformation, increasing eccentricity, and collapse that would occur in a simply supported column. As bending cracks occur and the passive reaction continues to increase, the eccentricity of thrust in the section actually decreases and the lining section continues to be able to take higher thrust. The resulting thrust capacity of the section is significantly greater than would be obtained for a simply supported column, particularly in the case of unreinforced sections. The level of the ultimate thrust that is finally reached is influenced by the amount of damage that takes place in the section as the lining deforms.

Although it has been recognized that a concrete tunnel lining can reach limiting moments without collapse, there has been little information on the maximum load levels that a lining can sustain under such conditions. In the research studies that have led to this report, large scale model studies and nonlinear analyses were carried out on concrete tunnel linings, both reinforced and nonreinforced, in ground having stiffnesses ranging from soft soil to hard rock. The studies concentrated on evaluation of the ultimate post cracking, nonlinear behavior of the concrete, which is responsible for capacities significantly higher than those obtained from elastic analyses. Although emphasis was placed on behavior of the concrete, an effort was made to model and evaluate a realistic range of soil and rock stiffnesses and loading conditions. The results of the study have provided a means of determining the ultimate thrust capacity of concrete linings for a range of loading patterns and flexibility ratios, representing the relative flexibility of the lining with respect to the soil or rock medium.

From the relationships presented in this chapter, with the flexibility ratio and the loading condition determined, or the initial elastic eccentricity of the thrust in critical lining sections decided, it is possible to estimate ultimate thrust levels as well as the tensile strains and cracking that can be expected below ultimate level.

Because the concrete lining is often placed after ground loads have stabilized and a nominal lining is capable of performing satisfactorily regardless of the existing loading conditions in many ground conditions, it is not necessary, nor is it universal practice, to carry out structural analyses for all concrete linings. However, structural analyses are useful in many situations, and designers do conduct such analyses. In some cases results are obtained that can lead to excessive reinforcement or overly thick sections, because of the inability to adequately quantify the true nonlinear behavior of the concrete as it approaches altimate load.

The research studies in this report have been directed toward evaluation of these nonlinear characteristics of concrete linings. The remainder of this chapter describes recommended approaches for evaluating the required structural capacity of a concrete tunnel lining, according to the following outline:

1. Determination of rock and soil loading conditions.
2. Selection of load factors to be applied to expected rock and soil loadings in order to ensure that actual loads fall in an acceptable working load range. Specific values of load factors chosen will depend on the conservatism in the assumed loading conditions.
3. Determination of the moment-thrust interaction diagram for evaluating the ultimate capacity of a concrete section, and

reduction of this capacity to account for uncertainties in strength of the materials and other factors as outlined by the ACI Code.

4. Determination of the moment-thrust path (eccentricity) at critical sections of the lining due to application of the soil or rock loads. Several analysis procedures based on linear assumptions are described in the report and others are available. The path is largely dependent on the flexibility ratio and the loading pattern.
5. Determination of the ultimate thrust capacity and corresponding load for a given moment-thrust path determined in 4 (If a nonlinear analysis was used in 4, then the ultimate thrust can be obtained directly from the nonlinear path). The charts presented in this chapter, based on the model tests and nonlinear analyses of the lining, permit the ultimate thrust capacity to be determined from either known flexibility ratios and loading conditions or from initial eccentricities determined from the linear beam-spring analyses commonly used by designers.
6. Evaluation of tensile strains at working loads and determination of reinforcement requirements. The load factors should be great enough to ensure that the actual loadings on the lining are in a range where cracking and deformations are not severe at service loads. Reinforcement requirements for these and other conditions are discussed.
7. Applicability of the ACI Code to tunnels. Sections of the ACI Code applicable to tunnels are noted and recommendations for utilization of the code for tunnel linings are made.

4.2 GROUND LOADS AND BOUNDARY CONDITIONS FOR ANALYSIS

Ground loads depend not only on the geologic conditions at the site and properties of the soil or rock, but also on the time and manner of installation of the lining, presence of other support, such as initial support, and assumptions regarding loading to be carried by initial and final support, additional loads occurring over the long term due to time-dependent effects or due to subsequent changes in loading (removal of air pressure, build up of ground water pressure, added fill, nearby excavation, etc.). Procedures for evaluating ground loads are discussed in Section 1.2.

4.2.1 Soil

Sandy Soil: Linings for tunnels in sandy soils are usually designed for pressures that are a function of the width of the opening. Most analyses, model tests and field measurements show that the load that develops is less than the equivalent of load due to the weight of soil extending /2 to 2 diameters above the tunnel crown. To determine the eccentricity, gravity loading can be used. If a beam-spring model is used for the analysis, tangential springs should be included in the active load region as well as where the radial springs are required.

Clay: Pressures that ultimately develop in soft, squeezing clays at shallow depth are some function of overburden pressure. Measurements show that lining pressures will increase with time (Peck, 1979), and approach overburden pressure for some clays. Peck recommended using overburden pressure for soft clays and pressures of $p_v\ (1 + Ko\ )/2$ for clays with high lateral stresses, where p_v is the vertical soil pressure at the tunnel level.

An approximation of the load distribution and the resulting eccentricity can be made using the elastic excavation loading analysis, assuming reasonable values for K_o . Time dependent and nonlinear soil models would more closely approximate the soil behavior.

Stiff Clays: Pressures may be as described for either sandy soil or soft clay, depending on soil stiffness, creep and the stiffness of the lining and its time of installatin. In most cases, tunnels at shallow depth (less than 100 ft.) in stiff soils are capable of taking full overburden pressure, using a nominal 8 to 12 in. thick (200 to 300 mm) concrete lining.

4.2.2 Rock

Loosening: Pressures will principally be a function of the weight of the wedges that can loosen immediately around the opening. Eccentricities can be determined from the loosening load analyses for various load distributions. Concentrated loads (due to small wedges) will produce higher eccentricities, but will produce lower pressures than will loadings form large wedges, in which eccentricities are smaller but pressures are greater. Both tangential and radial springs should be employed if a beam-spring model is used. In most rock tunnels, eccentricities will be low enough to cause the elastic moment-thrust path to intersect the envelope above the balance point. Thus, linear analyses will provide a reasonably accurate estimate of ultimate lining capacity.

Squeezing: High ground loads may develop on the lining system, although the final lining, if installed after most of the movements have taken place will be subject to only small pressures.

Final Concrete Linings: The loadings described for soil and rock, although developing on the total lining system, may not fully act on the final concrete lining installed at some later time and supported initially by another system. The loads due to grouting, water pressure and other time-dependent or delayed loading are discussed in more detail in Section 1.2. Not only may loads on the final lining be reduced, but eccentricities may also be smaller.

4.3 LOAD FACTORS

In ultimate strength design of concrete members, the design load to be applied to the member is multiplied by a load factor, a quantity greater than 1. The member is then designed to reach its ultimate capacity when the factored load is applied. The load factor combined with the capacity reduction factor provides a safety factor. The safety factor is necessary to prevent excessive creep of the concrete, possible failure of concrete due to long term loading excessive cracking, local spalling due to stress concentrations and to account for uncertainty in material properties and analysis procedures. It also is designed to guard against overload. However, the fact that a major part of the safety factor is provided by applying a load factor should not be interpreted as applying the entire load factor to the uncertainty in loads; part of the load factor is necessary to maintian stresses and distortions in the concrete at an acceptable level when the service ground loads are present.

Although values of load factor are specified in the ACI Code, values are not recommended in this report because of the varying degrees of conservatism often built into the evaluation of ground loads in underground engineering practice. For example, in some cases, the ground loads assumed to act on a lining may represent an upper limit or an envelope of the possible loadings on the structure. In other cases, all ground loads may be assumed to be applied to the concrete lining, even though it is known that the initial support (which carried the initial loads) will remain in place and maintain much of its capacity, particularly when encased in concrete. In cases such as this, where the ground load has been conservatively estimated, the value of the load factor to be used would be less than the value required for use where the levels of the assumed ground loads are actually expected to develop on the lining. If the design ground load represents an uppper limit to the possible loadings on the lining, then it is not necessary to include, as part of the load factor, a quantity that accounts for the uncertainty in the loading, but it would still be necessary to have a value of the load factor sufficient to cause the stresses and distortions in the structure to be at an acceptable, working level when the ground loads act on the lining.

For reference, load factors in the ACI Code for builings are 1.4 for dead loads, which are known with some certainty. If the capacity reduction factor is 0.7, then the corresponding safety factor is 2.0 (i.e., 1.4/.7); the stress in the concrete will then be about 1/2 f'_c under the selected loads. Higher values of 1.7 are used for live loads that are considered to be known with less certainty. Load factors applied in the design of permanent steel-shotcrete linings used as both initial and final structural lining and installed close to the face in 60 to 70-ft-wide shallow chambers excavated in rock are given as examples of what has been used. A load factor of 2 was applied to the load due to rock wedges. These wedges were observed to form, and measured loads on the linings were in the range of the loading calculated for them. The height of the rock load above the crown was typically less than 1/2 the width of the opening. A load factor of 1.2 was applied to the full overburden load (on the order of 80 to 100 ft of rock and soil). This loading was an upper limit, and it is probable that, under the worst case, if the rock mass over the tunnel had been allowed to loosen and collapse onto the lining, that frictional effects would prevent the most severe loading from exceeding approximately 70% of the overburden stress.

4.4 SECTION CAPACITY

Moment-thrust combinations obtained from the factored loads are most easily checked by comparing them with the moment-thrust failure envelope or interaction diagram. Use of the ACI Code (ACI 318-77) procedure for construction of the diagram is recommended. This procedure contains capacity reduction factors that provide additional safety to account for uncertainty in material properties, calculation of the resistance of the section, and the difference between concrete strength obtained from cylinder tests and the concrete in the structure. A capacity reduction factor of 0.7 for columns that gradually becomes 0.9 for pure flexure is applied with a transition near the pure flexure value.

The capacity reduction factors suggested are the same for linings as for structures covered by the ACI Code because there appears to be little reason why the uncertainties that are taken into account by these factors should be grossly different, and the load factors should be adjusted to provide the desired overall safety factor. Better curing conditions in a tunnel may provide a stronger concrete, but this advantage may be offset by the difficult conditions under which it is often placed by pumping.

Shear resulting from the analysis may be compared directly with the shear strength calculated from Section 11.3 of ACI 318-77 which takes into account the effect of thrust. If the part of the lining for which the shear is checked is near a corner or knee of an arch that may be considered a support for the member, the shear should be checked at a distance equal to the effective depth from the face of the support. If there is no such support as in a circular or arch tunnel, shear should be checked at the point of its maximum value. Shear strength of embedded steel supports may be added to that of the concrete sections. It would be normally unreasonable to provide shear reinforcement such as stirrups in a tunnel lining, and therefore the thickness would normally be adjusted to resist shear if needed. If a rock block or wedge moves, high shear forces will be concentrated at its edges and if the movement is significant, a shear failure may occur in the lining; if the rock block is moving parallel to the discontinuity, the movement may cause shear forces along the discontinuity to build up and the rock block will not dislodge, but a shear failure may occur in the lining that should be avoided; if the rock block movement has a component normal to the discontinuity, the shear forces will not increase during movement (and may reduce to zero) so the lining must resist the movement in shear to prevent a drop out. Fairly concentrated loads, as may result from small wedges of loosening rock, cause high shear at their edges, and this condition combined with a relatively thin lining could lead to a possible shear failure and should be checked.

4.5 MOMENT-THRUST PATH

The relative stiffness of the lining with respect to the soil or rock is expressed for flexure problems by the flexibility ratio,

$$F = \frac{E_m R^3}{6E_\ell I_\ell}\left[\frac{2(1 - \nu_\ell^2)}{(1 + \nu_m)}\right]$$

where E_m and E_ℓ are the elastic moduli of the medium and lining, I_ℓ the moment of inertia of the lining, and R the mean radius. This ratio largely controls the eccentricity (ratio between moment and thrust) developed at critical sections in the lining. Linear analysis procedures, both closed form and beam-spring stress analyses, are available and have been routinely used by designers in evaluating the thrust and moment developed in a lining for various loading configurations. Such analyses, summarized in Section 1.3, have major limitations when used to evaluate the required capacity of concrete linings in conditions where the eccentricity due to the load is large enough to produce significant tension in the concrete lining. Use of linear analyses in design has in some cases led to the use of excessive reinforcement or lining thickness to resist the computed tension. Such an approach does not recognize the capability of an unreinforced concrete arch to crack when in contact with the ground, yet still carry appreciable thrust, without excessive distortion in a section having no tensile capactity.

The model studies and nonlinear parameter studies summarized in this report have provided a means for evaluating the ultimate capacity of such linings. In these analyses the soil stiffness is assumed linear, but the concrete lining is treated as a nonlinear, section, with moments that decrease beyond certain rotations. Figure 4.1 shows the differnce that would be obtained between the linear and nonlinear analyses. In the nonlinear analysis, when the moment-thrust (M-T) path approaches the interaction diagram (envelope) below the balance point, the concrete cracks and the eccentricity decreases, resulting in a

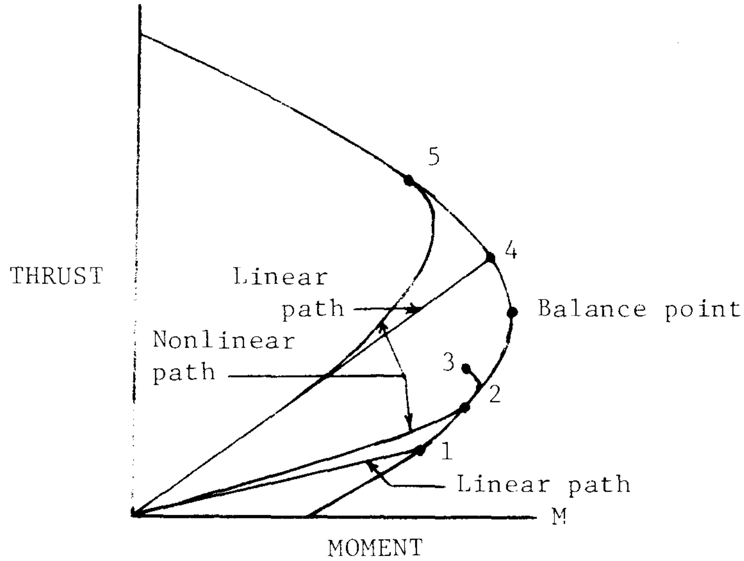


FIGURE 4.1 LINEAR AND NONLINEAR M-T PATHS AND PREDICTION OF FAILURE

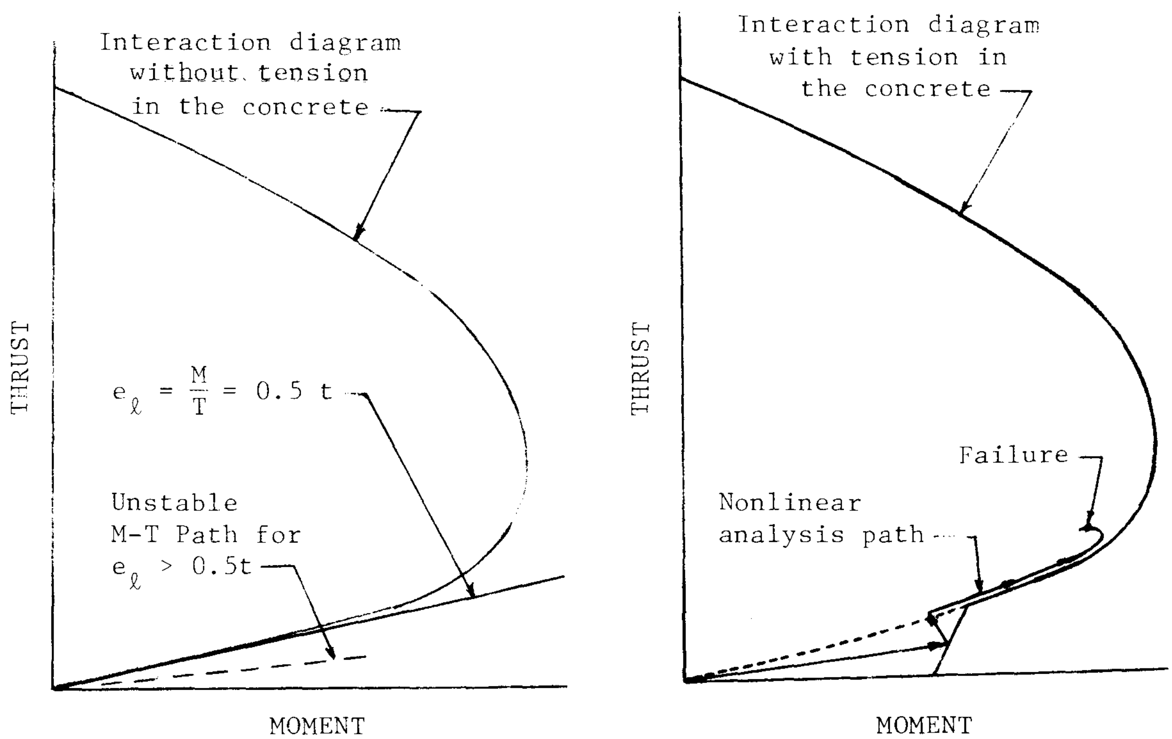


FIGURE 4.2 BEHAVIOR OF AN UNREINFORCED SECTION WITH $e_\ell > 0.5t$

higher value of thrust (point 2) than would be obtained in the linear analysis when the M-T load path intersects the M-T envelope (point 1). The section has further capacity even after the load path has reached the envelope, and the thrust continues to increase even though the moment capacity drops off (point 3). The results of the parameter studies and the large scale model tests show that the concrete strains begin to increase dramatically once the curve breaks away from the envelope toward the thrust axis. Typically, compressive strains at the section are in the range of 0.003 when the envelope is reached (Figure 3.31), but is much larger when failure actually occurs. For reinforced concrete sections, at the point where failure occurs the thrust value for the nonlinear case is typically 1.8 to 2.8 times the ultimate thrust estimated from the linear analysis.

An unreinforced lining in a soft medium presents, special problems when the eccentricity is larger than 0.5 t and tension in the concrete is ignored in constructing the interaction diagram. In this case, the M-T path will fall below the interaction envelope at the start of loading as shown in Figure 4.2. A linear analysis would lead to the conclusion that reinforcement is required to prevent failure of the section. When tension cracking occurs, the path jumps to the cracked interaction diagram and follows it until failure occurs in thrust. When cracking occurs, the lining behaves as a series of unbolted segments with joints at the cracks and the failure envelope becomes the moment-thrust path. Failure would be predicted as soon as the path jumps back to the interaction diagram; however, the nonlinear analysis has shown that considerable additional strength is available as the M-T path follows along the M-T envelope until a curvature is reached that actually causes the section to disintegrate, moments fall off and the ultimate thrust capacity is reached. Thus, if reliance is placed on linear analyses for M-T load eccentricities greater than 0.5/t, reinforcement is required to produce a stable result, even though it is well known that such a condition does not represent collapse, nor does it necessarily produce excessive distortions and cracking.

Figures 4.3-4.5 show typical results of the nonlinear parameter studies for various loading configurations. The maximum pressure, P_u applied to the lining or the ultimate thrust, T_u, on the initial lining section is plotted as a function of the flexibility ratio in Figures 4.3 and 4.4 for a typical set of parameters. Additional results are shown in Figures 3.30-3.38. In the excavation loading case, the medium is assumed elastic and the lining thrusts and moments due to immediate installation of a lining upon tunnel excavation are determined using a linear finite element mesh for the continuum and joint elements between the medium and lining. The pressure, p_u, represents the total overburden pressure when the lining reaches its ultimate capacity. This analysis gives an actual pressure transmitted to the lining that is approximately 70 percent of the full overburden load. For the gravity load cases, the pressure p_u, is applied directly to the lining and then the interaction between lining and the soil or rock medium is determined using a beam element model for the lining and tangential springs to represent the soil or rock medium. The gravity load is equivalent to the so-called loosening pressure resulting from some height of rock or soil load, proportional to the width of the opening. Tangential springs are used around the entire perimeter with $K_t/K_r = 0.25$, and radial springs are used only where applied load causes the springs to be in compression. In Figure 4.5 all the analysis data is shown in terms of the thrust ratio T_u/T_o versus the ratio of linear eccentricity divided by the lining thickness.

4.6 ANALYSIS OF LINING AND COMPARISON WITH ULTIMATE CAPACITY

In evaluating a tunnel lining, the plots based on the nonlinear analyses developed in this report can be used directly to obtain the ultimate thrust, if the flexibility ratio is known and the lining shape (circular or arched) and loading patterns are similar to those used in the analyses. The plot in Figure 4.5 will also permit the ultimate nonlinear thrust capacity to be determined by first calculating the linear eccentricity calculated from a linear analysis. The value of

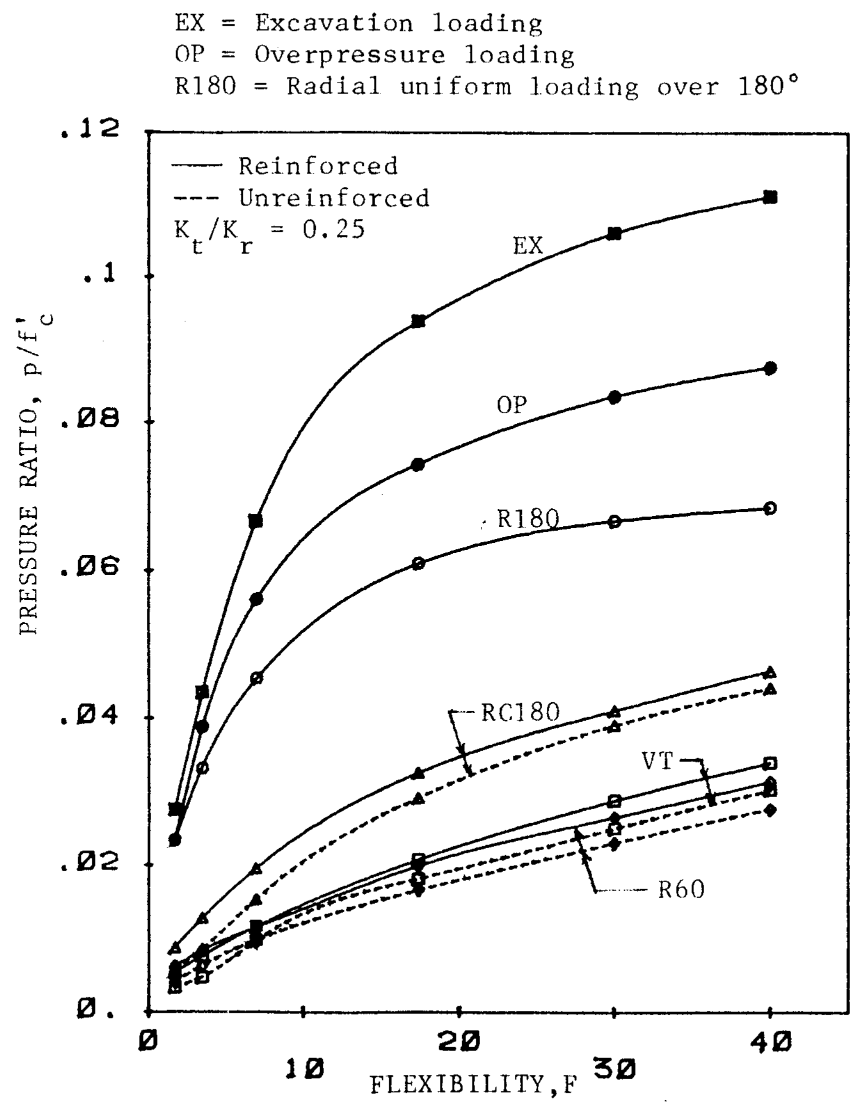


FIGURE 4.3 COMPARISON OF LOADING CONDITIONS AS FLEXIBILITY VARIES IN TERMS OF p/f'_c FOR GRAVITY LOADING AND K_t/K_r OF 0.25

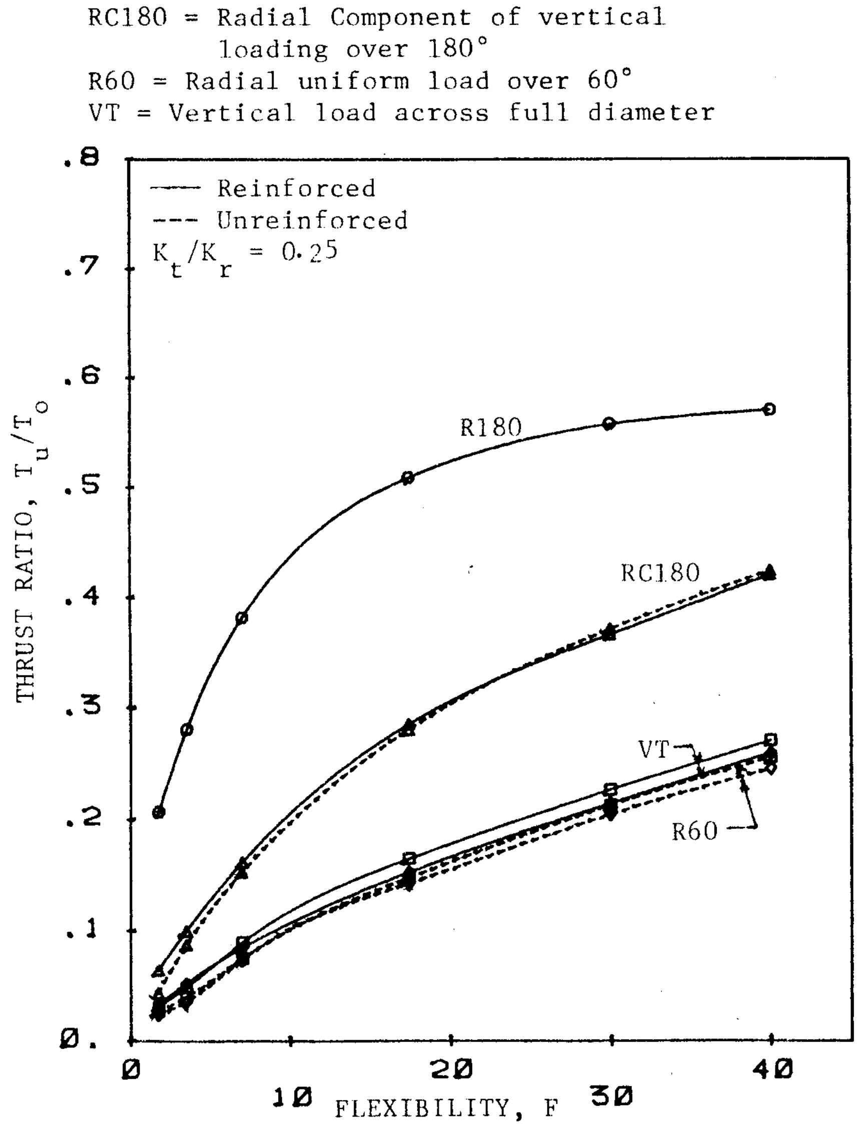


FIGURE 4.4 COMPARISON OF GRAVITY LOADING SHAPES AS FLEXIBILITY VARIES IN TERMS OF T_u/T_o AND FOR K_t/K_r OF 0.25

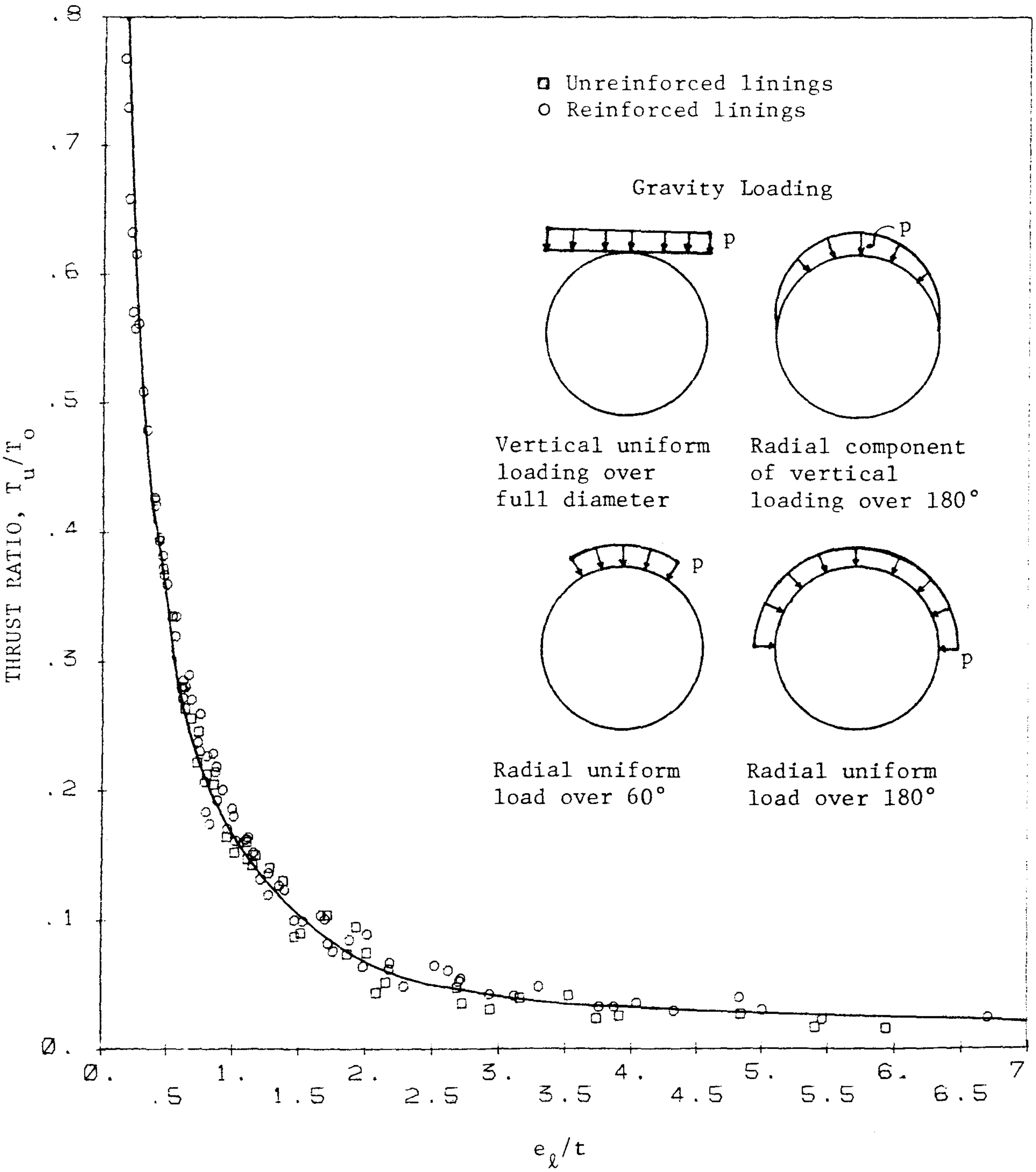


FIGURE 4.5 PREDICTION OF FAILURE OF LININGS BY THE NONLINEAR ANALYSIS AS A FUNCTION OF THE LINEAR ECCENTRICITY DIVIDED BY THICKNESS

the eccentricity from the linear analysis is principally a function of flexibility ratio, load patterns and lining configuration. This approach can be used when the lining configurations and loadings differ from those used in the nonlinear analyses presented in the report. The relationship between a given linear eccentricity and the ultimate nonlinear thrust capacity for a lining in contact with the soil or rock is principally a function of the properties of the lining section and the nonlinear characteristics of the concrete, and is not strongly influenced by flexibility ratio, loading pattern, or lining configuration. In other words, several different combinations of flexibility ratio, loading pattern, and lining size and shape will produce the same linear eccentricity. For all of these cases, the ultimate nonlinear thrust capacity would be almost the same, assuming that the concrete properties and section capacity were the same for each case. Thus, the curve in Figure 4.5 can be used to obtain the thrust at failure based on the computed eccentricity obtained from a linear analysis for gravity loading.

It is also possible to use the linear analyses to determine the eccentricity, but to modify the applied thrust levels from those that would be predicted from this analysis, based on previous experience and observations. For example, measured thrusts, such as those presented by Peck for soft clays (1969) could be used to determine the thrust level, $T_a =$ pR due to the applied pressure. The linear eccentricity would then be determined from an analysis, such as the overburden or excavation loading analysis. These cases, although they do not account for the plastic and creep behavior of the clay, give reasonable values of the linear eccentricity in the concrete section, if appropriate soil stiffnesses have been used, and the loading pattern is reasonable. The ultimate nonlinear thrust capacity would then be determined from the given eccentricity value, using Figure 4.5. The applied thrust, obtained from the empirical data, would be multiplied by an appropriate load factor. This value would then be compared with the ultimate thrust capacity represented by the interaction diagrams to ensure that it is less than the ultimate value.

Caution must be exercised in such approaches, because it is possible to select thrust levels inappropriate to the elastic eccentricities obtained from the analysis. In general, the higher the thrust level, the more uniformly distributed the pressure must be, and the lower the eccentricity. For linings in continuous contact with the rock or soil, extremely high eccentricities can only occur for relatively small, concentrated loads.

It is recommended that the gravity load case for soils be used with pressures that are equivalent to a height of soil load of less than approximately two times the tunnel diameter.

In summary, to determine the maximum allowable thrust level for a given concrete lining, the following is recommended:

1. Below the Balance Point
   a. Use a nonlinear analysis to determine where the nonlinear M-T path intersects the reduced M-T envelope. The reduced M-T envelope should be drawn using the recommended factors in the ACI Code to account for concrete strength variations. Verify that the applied thrust, T_a, times the load factor is less than the ultimate thrust, T_u reduced by the capacity reduction factor.
   b. Alternatively a linear analysis may be used to determine the linear eccentricity of the thrust for the given flexibility and loading conditions, and then the chart of Figure 4.5 may be used to obtain the T_u (nonlinear); this value is then compared with the applied T_a times an appropriate load factor.
2. Above the Balance Point

The same procedures outlined in (1), above, can be utilized in evaluating the capacity of a lining where the linear moment-thrust path intersects the envelope above the balance point. However, above the balance point, the ultimate thrusts calculated from the linear analysis will be closer to that obtained with a nonlinear analysis than it is below the balance point, so the linear analysis can be used directly without being excessively conservative.

For most linings in rock, and for linings in many of the stiffer soils, the moment-thrust path will approach the envelope above the balance point so the linear analysis can be used.

4.7 EVALUATION OF STRAINS AND CRACKING

The actual thrust levels determined from the assumed loads (without load factors) should fall low enough along the moment-thrust path that distortions and tensile strains do not cause excessive damage.

Concrete will creep in tension as well as compression so if the loads are applied slowly, larger tensile strains can occur without a crack forming. Therefore, for a final lining placed in a presupported opening where considerable time will be required for the loads to reach the final lining it is reasonable to double the allowable cracking strain when a cracking analysis is performed. A reasonable value of allowable tension strain to avoid cracking from this criteria would then be on the order of 0.0003 in./in. Figures 4.6 and 4.7 from the nonlinear analyses show the level of tensile strains developed in both unreinforced and reinforced concrete sections for typical cases of gravity loading in a soft media. Maximum tensile strains of 0.001 in./in., at the center of a 90 degree arc of the lining and dropping to zero at the edges of the arc would produce a total width of a single crack of approximately 0.1 in. (two.5 mm) for a 10-ft-radius (3.0 m) lining. If longitudinal cracks were spaced 12 in. (300 mm) on center, the maximum crack width would be close to the 0.01 in. crack level, recognized as a working limit in some concrete pipe design. The 0.0003 strain level, assumed to be the initiation of cracks, would produce crack widths exceeding the 0.01 in. (0.25 mm) level only for crack spacings greater than approximately 3 ft. Concentration of the deformation could occur in a few cracks for a non-reinforced lining, causing an increase in the width of the individual cracks. This condition would be more likely to occur where concrete lining thickness is variable around the perimeter, perhaps due to overbreak, and where

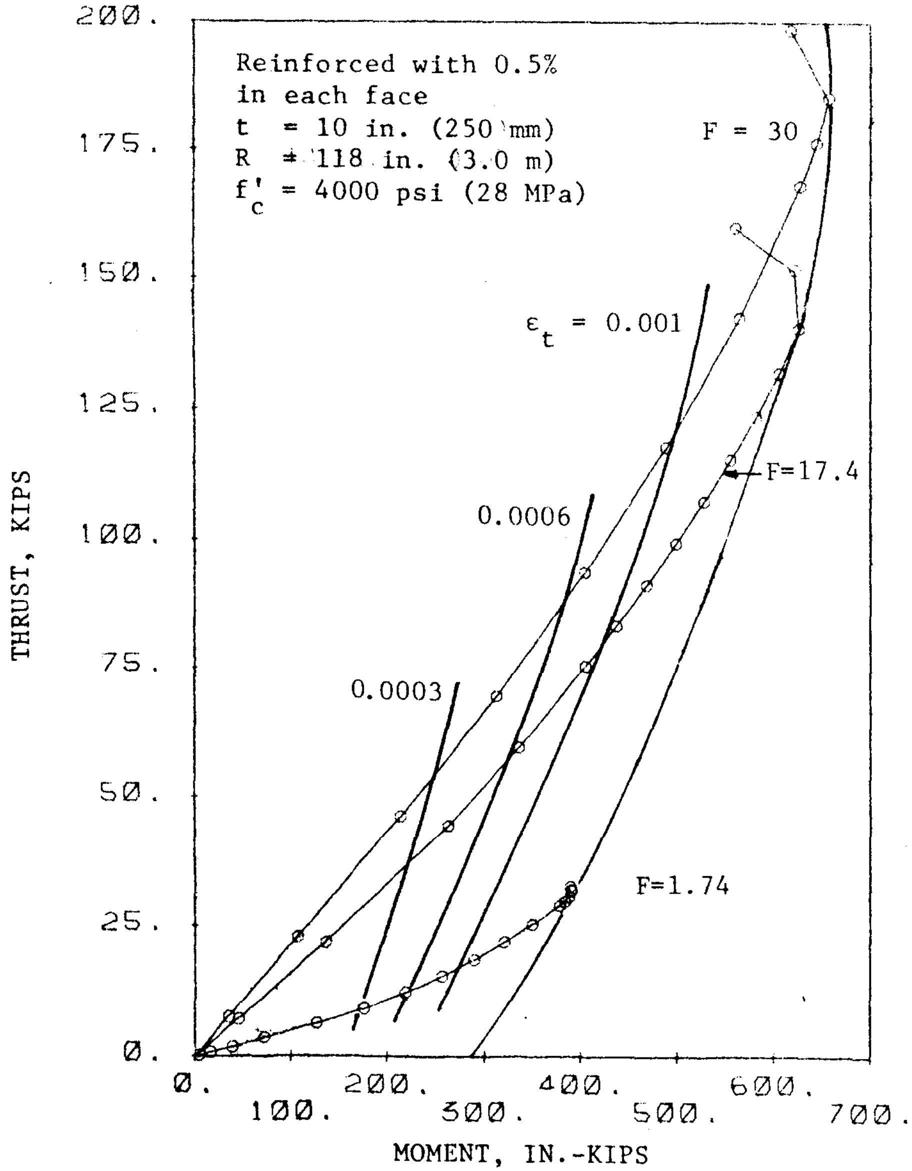


FIGURE 4.6 TENSION STRAINS ALONG THE M-T PATH FOR A REINFORCED LINING

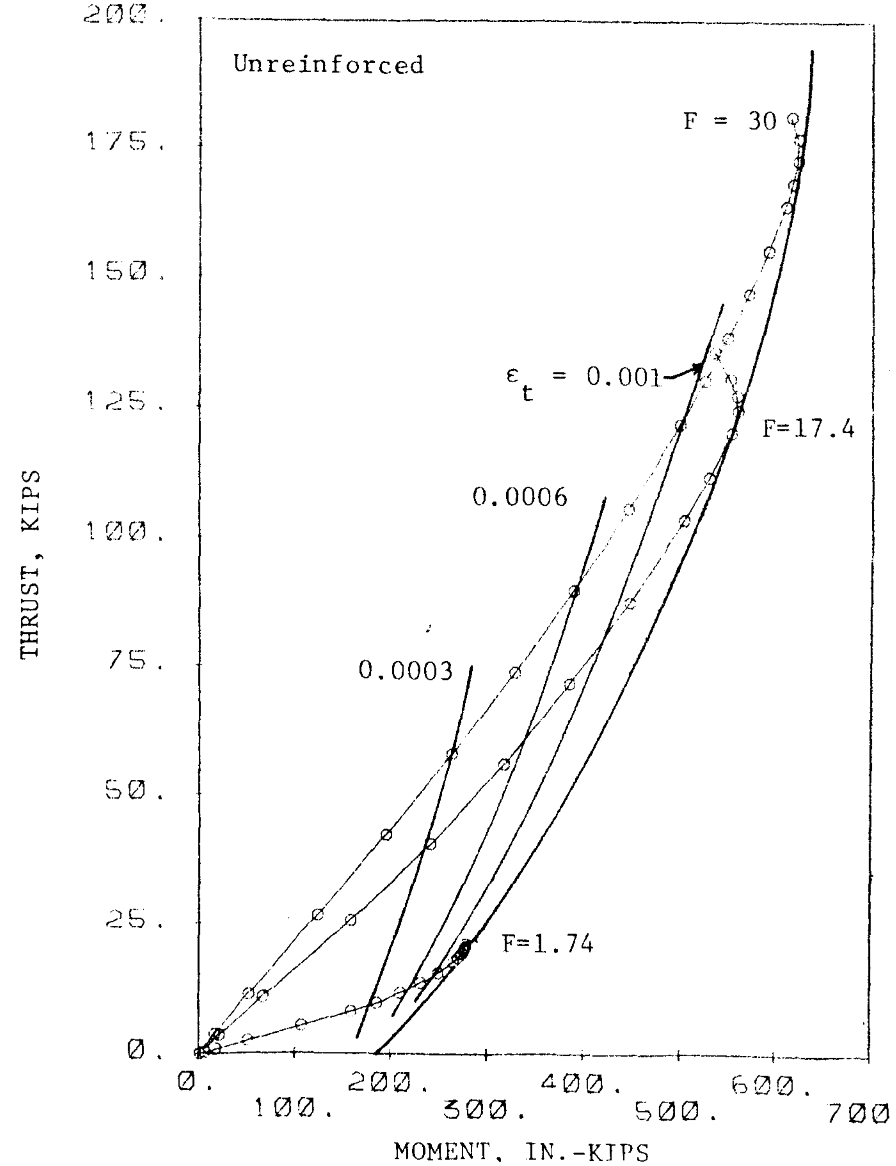


FIGURE 4.7 TENSION STRAINS ALONG THE M-T PATH FOR AN UNREINFORCED LINING

lining-to-rock or lining-to-soil contact is poor. Shrinkage and temperature effects would influence cracking, also. Usually the most pronounced cracking occurs due to shrinkage, often forming in the circumferential direction.

To ensure that cracking at the 0.001 in./in. tensile strain level is not excessive, a light reinforcement may be needed to spread cracks. At the 0.0003 in./in. strain level, crack widths should not be excessive due to lining distortion, even for non-reinforced sections.

The model tests discussed in Section 3.1 shows that reinforcement in a lining does little for strength in the amounts usually used for underground supports. However, reinforcement is effective in distributing cracks and therefore keeping each crack from becoming large. When no reinforcement was used in the model tests, only one crack would form in each high moment region, but if reinforcement was present, the cracks would occur 4 to 6 in. (102 to 152 mm) apart and would be much finer. On the other hand the tests also showed that cracks first appeared at loads 50 percent or greater than the ultimate load. This percentage would be larger if the loading was appplied very slowly. Therefore, flexure cracks are not common at working loads and flexure reinforcement is not recommended to control them unless there is a definite need shown by calculations. If cracking appears to be a problem, a minimum amount of reinforcement of from 0.25 to 0.50 percent should be used on the tension side of the highly stressed region, but is not necessary throughout the lining.

The main reason proposed for limiting flexureal cracking in a lining is to prevent leakage. However, when a crack occurs a substantial area of the section is still in compression, and the stress becomes larger as the area is reduced. This compressed concrete should remain an effective barrier to water penetration. On the other hand, circumferential cracks due to shrinkage penetrate through the section and become a more likely path for leakage. The longitudinal flexural cracks due to ground loads are probably much less a problem insofar as leakage is concerned than shrinkage cracks, joints or casting flaws that might occur.

4.8 APPLICATION OF ACI CODE

Many provisions of ACI 318-77 do not apply to underground construction, others require modification, and a few may be used without change; Chapter 4 "Concrete Quality" and Chapter 5 "Mixing and Placing Concrete" describe standard requirements for preparing and placing concrete and in general, apply to underground work. Quality concrete without flaws is required for strength and to prevent leakage. The transit time without agitation is sometimes longer in tunnel work than above ground and therefore larger slumps are required at the mix plant to assure workability at placement. At placement the Code requires that "Concrete shall be deposited as nearly as practicable in its final position to avoid segregation due to rehandling or flowing." Since the concrete for linings is normally pumped into the cavity behind the linings at the crown or through windows at the springlines, a considerable amount of flowing is required to get concrete distributed around the forms; techniques have been developed in the tunneling industry to reduce segregation in this case and a satisfactory placement is attained even when there is considerable flowing of the concrete. Additives such as the various superplasticizers have helped greatly in this regard by improving strength while also improving workability without segregation.

Chapter 6 "Formwork, Embedded Pipes, and Construction Joints" has two sections that apply to cast linings. Section 6.2 "Removal of Forms and Shores" is adequate in principle, because it states that forms may not be removed until the structure and any remaining shoring can support its weight and any loads that may be on it. Since the ground is stable when the final lining is placed, either due to the use of initial supports or the integrity of the ground, then the only load that should occur is the weight of the concrete lining itself. With this self weight applied to the lining, an analysis to determine maximum concrete stresses, with allowances in the analysis for interaction with the ground, may be performed. Once the maximum stresses are known, the acceptable compressive stress to be reached by the concrete must be selected.

At an early age, concrete tends to creep more rapidly under load than it does when fully cured. Creep may allow deformation of the lining and separation from the ground at the crown if the forms are removed prematurely. Normally deformations will be negligible if the concrete strength has reached four times the maximum compressive stress in the concrete. With the required compressive strength of the concrete determined, it is necessary to monitor the gain of strength in the lining with time, to determine when the required strength has been reached. This may be done with field cured cylinders or by other means. However, the rate of strength gain is influenced by many things such as water content, temperature, cement formulation and fineness of grind. Therefore, the time at which a given strength is reached may change suddenly without an obvious cause, and close monitoring is required. This approach to removal of forms allows greater latitude to the contractor in his casting operation and provides the opportunity to increase the casting rate by increasing the strength gain of the concrete through the use of additives or more rapid set cement types.

Construction joints are necessary to separate castings when the operation cannot be continuous. Specially prepared vertical joints are often specified and are rather expensive because of the labor and time required for their placement. It is proposed that a sloping joint in which the concrete is allowed to assume its own angle of repose will be adequate, and is no more likely to leak than a vertical joint without a special water stop. If separation of the joint occurs due to shrinkage of the concrete, the opening is actually less for a sloping joint than for a vertical one. When this type of joint is used, the surfaces should be prepared as described in Section 6.4 of ACI 318-77 before the next pumping cycle.

Chapter 7 "Details of Reinforcement" contains provisions for protection of reinforcement and to assure that it is effective when used. Therefore, the sections on bending and surface conditions apply. The tolerances on placement may be relaxed because of the difficulty in placing bars, but the specified cover should be increased to compensate for the changes in tolerance and greater tendency to rust. Concrete cover over reinforcement is specified as 3 in. (76 mm) for concrete cast against the ground and appears appropriate for cast in place tunnel construction. Reinforcement on the inside face should have at least 2 in. (50 mm) of cover as required by the ACI Code for construction exposed to the weather. Though the tunnel surface is not actually exposed to the outside weather it may be exposed to frequent wetting due to condensation on the surface that will eventually reach the inside bars. In many old tunnels much damage has occurred due to spalling of concrete due to expansion of corroding bars that exposed the bars and accelerated the process. In precast construction where the concrete is placed with greater control and is of higher quality, it may be possible to further decrease the cover without causing long term corrosion.

At the crown where inside reinforcement is in tension, there is an inward force on the concrete around the bars when moment occurs and the region flattens, or the radius of curvature is increased. The cover in this region should be checked to assure that the concrete tension force around the bars is adequate to hold them in against the radial force that results when tension tries to straighten the bar. If the diagonal tension in the concrete along the bar is inadequate, it may be necessary to increase the cover in this region. The crown region of the lining is sometimes flattened on the inside to allow additional room for the pumping slick line to be inserted above the form. This practice eliminates the problem discussed above and also increases the bending strength of the lining in the crown region.

The sections on lateral confinement of bars in compression by ties and stirrups are not appplicable. Horizontal confinement in the longitudinal direction of the tunnel is not necessary because the lining is continuous in this direction. Confinement outward is provided by the rock or soil on the outside of the lining. Also the inside bars are in tension in the crown region, and on the sides where they are in compression, the original curvature is outward so their tendency to buckle is toward the inside of the member (outside of the tunnel) where confinement is adequate.

Chapter 9 "Strength and Serviceability Requirements" describes the load factors, capacity reduction factors and control of deflection and cracking that were discussed previously. Various considerations in the control of cracking are discussed by ACI Committee 224 (ACI-224, 1972) where means are provided for estimating crack size and distribution due to shrinkage and flexure. The formulas presented there for maximum crack width due to flexure are written in terms of the tension reinforcement stress and in this form can be applied to members with axial load, as well as flexure. The axial compression results in a uniform stress that reduces the flexural cracking by reducing the tension stress in both the concrete and tension reinforcement. Use of these formulas will result in excessive additional reinforcement when used with 3 in. (76 mm) of cover, however, because they were devised for slabs and framed walls with less than 2 in. (51 mm) of cover. Recognizing this problem, CRSI Bulletin PSI-7804A recommends using a maximum value of 2 in. (51 mm) for cover in crack calculations when the actual cover is greater than 2.0 in. (51 mm). Also crack width normally need only to be checked on the outside face for most transportation tunnels in non-public places.

Chapter 10 "Flexure and Axial Loads" concerns the calculation of strength and this topic has been discussed. The methods for calculating strength presented there have been recommended. The limit on minimum flexural reinforcement in this chapter is to assure that the ultimate flexural strength is larger than the cracking strength to avoid failure upon cracking. This provision does not apply to linings because failure would not occur at cracking since the force within the lining would be redistributed as a result of the high degree of redundancy and confinement of the ground.

Chapter 11 "Shear and Torsion" can be applied to calculate the shear strength of linings as described in Section 11.3 where provisions are included for considering the axial force effects on shear.

Chapter 12 "Development and Splices of Reinforcement" should be applied to lining design in order to assure that needed reinforcement will be effective. Development of bar forces are primarily a function of the bar geometry and concrete strength, and therefore does not change for underground structures.

4.9 SUMMARY

The recommendations given in the preceding sections were developed as a result of analyses and model testing of the behavior of concrete tunnel linings. The research addressed problem areas in current design practice, and the results have provided insight into the areas of uncertainty that have led designers to overconservatism in tunnel lining design.

The recommended procedures provide sufficient latitude for designers to exercise judgment gained through experience and allow the flexibility required by site-specific conditions. Details of the suggested approach are based upon procedures that have been accepted for years in the design of above-ground structures, with appropriate modification to capitalize on the benefits of ground/structure interaction.

BIBLIOGRAPHY

ACI Committee 224, (1972), "Control of Cracking in Concrete Structures," Journal of the American Concrete Institute, Vol. 69, No. 12, pp. 717-757.

ACI Committee 318, (1977), "Building Code Requirements for Reinforced Concrete," ACI Standard 318-77, American Concrete Institute, Detroit, Michigan.

Barton, N., R. Lien and J. Lunde, (1974), "Engineering Classification of Rock Masses for the Design of Tunnel Support," Rock Mechanics, Vol. 6, No. 4, pp. 189-236.

Bienawski, Z.T. (1974), "Geomechanics Classification of Rock Masses and its Application in Tunneling," Third International Conference on Rock Mechanics, Denver, Colorado, Vol. 11a, pp. 27-32.

Burns, J.Q. and R.M. Richard, (1964), "Attenuation of Stresses for Buried Cylinders," Proceedings, Symposium on Soil-Structure Interaction, Tucson, pp. 378-392.

Cording, E.J. and J.W. Mahar, (1978), "Index Properties and Observations for Design of Chambers in Rock," Engineering Geology, No. 12, pp. 113-142.

Curtis, D.J., (1976), Discussion of: Muir Wood, A.M., "The Circular Tunnel in Elastic Ground," Geotechnique, Vol. 26, No. 2, pp. 231-237.

Dar, S.M. and R.C. Bates, (1974), "Stress Analysis of Hollow Cylindrical Inclusions," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT2, pp. 123-138.

Dixon, J.D., (1971), Analysis of Tunnel Support Structures with Consideration of Support-Rock Interaction, Report of Investigations 7526, U.S. Bureau of Mines, Washington, D.C.

Einstein, H.H., C.W. Schwart, W. Steiner, M. Baligh, and R. Levitt, (1980), "Improved Design For Tunnel Supports: Analysis Methods and Ground Structure Behavior: A Review-Volume II," Report No. DOT/RSPA/DPB-50/79/10, U. S. Department of Transportation, Research and Special Programs Administration, Office of University Research, May.

Einstein, H.H., W. Steiner and G.B. Baecher, (1979), "Assessment of Empirical Design Methods for Tunnels in Rock," Proc. RETC, Vol. 1, Chapter 40, AIME-ASCE, pp. 683-706.

Hoeg, K., (1968), "Stresses Against Underground Structural Cylinders," Journal of the Soil Mechanics and Foundation Division, p321, Vol. 94, No. SM4, pp. 833-858.

Morgan, H.D., (1961), "A Contribution to the Analysis of Stress in a Circular Tunnel," Geotechnique, Vol. 11, March, pp. 37-46.

Muir Wood, A.M., (1975), "The Circular Tunnel in Elastic Ground," Geotechnique, Vol. 25, No. 1, pp. 115-127.

Peck, R.B., (1969), "Deep Excavations and Tunneling in Soft Ground," Proceedings, Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico City, State of the Art Volume, pp. 225-290.

Peck, R.B., A.J. Hendron, Jr. and B. Mohraz, (1972), "State of the Art of Soft Ground Tunneling," Proc., First North American Rapid Excavation and Tunneling Conference, Chicago, AIME, Vol. 1, pp. 259-286.

Proctor, R.V. and T.L. White, (1969), "Rock Tunneling with Steel Supports," The Commercial Shearing and Stamping Co., Youngstown, Ohio.

Ranken, R.E., J. Ghaboussi and A.J. Hendron, Jr., (1978), Analysis of Ground-Liner Interaction for Tunnels, Report No. UMTA-IL-06-0043-78-3, Urban Mass Transportation Administration, U.S. Department of Transportation, Washington, D.C., October.

Saha, P.K., (1982), Finite Element Modeling and Structural Behavior of Concrete Tunnel Linings, Ph.D. Thesis, University of Illinois, Urbana, 235 p.

Schwartz, C.W., and H.H. Einstein, (1980), "Improved Design of Tunnel Supports: Volume I-Simplified Analysis for Ground-Structure Interaction in Tunneling," Report No. UMTA-OMA-06-0100-80-4, U. S. Department of Transportation, Research and Special Programs Administration, Office of University Research, May.

Sgouros, G. E., (1982), Structural Behavior and Design Implications of Concrete Tunnel Linings Based on Model Tests and Parameter Studies, Ph.D. Thesis, University of Illinois, Urbana, 385 p.

Steiner, W., H.H. Einstein, (1980), "Improved Design of Tunnel Supports: Volume 5-Empirical Methods in Rock Tunneling--Review and Recommendations," Report No. UMTA-MA-06-0100-80-8, U.S. Department of Transportation, Urban Mass Transportation Administration, Office of Rail and Construction Technology, June.

Terzaghi, K., (1946) Revised (1968), "Rock Defects and Loads on Tunnel Supports," in Proctor and White, Rock Tunneling with Steel Supports, Youngstown, Ohio, pp. 17-102.

Wickham, G.E., H.R. Tiedemann and E.H. Skinner, (1974), "Ground Support Prediction Model-RSR Concept," Proceedings Second RETC, San Francisco, Vol. 1, pp. 691-707.

☆U. S. GOVERNMENT PRINTING OFFICE: 1983--700-497--247

450 copies