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The University of Alberta

Tunnel Design in the City of Edmonton

by

© Christel MATHIS

A Thesis

submitted to the Faculty of Graduate Studies and

Research in partial fulfillment of the requirements

for the degree of Master of Science.

Department of Civil Engineering

Edmonton, Alberta

Fall, 1974

The University of Alberta

Faculty of Graduate Studies and Research

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research for acceptance, a thesis entitled "TUNNEL DESIGN IN THE CITY OF EDMONTON", submitted by Christel MATHIS in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

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Abstract

Due to the advances in the knowledge of tunneling in soft ground, a review of the design of the Edmonton tunnels has been undertaken.

The well-known polygonal method of Anders Bull has been selected to calculate moments, thrusts and deflections of a lining under a given symmetrical loading. The method has been programmed in order to provide a convenient tool for the design of tunnels in an elastic homogeneous medium.

The crucial factors governing the lining behaviour have been evidenced by parametric study. The rib spacing and the concrete thickness are the critical factors determining the safety of the lining and the design is governed by the value of the soil reaction coefficient, the extent of arching, and the reliability in the long term of the temporary support as part of the permanent lining. The latter factors are as yet largely unknown and a definite conclusion on the safety of the design cannot be drawn until more field data is available.

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Chapter I

Introduction

In 1955, the City of Edmonton decided to improve its waste treatment facilities. The project called for the construction of an extensive collecting system of sewer interceptors connected to a centrally located Waste Treatment Plant.

Numerous advances in the understanding of tunnel behaviour have been made since 1955 so that a review of the design of the Edmonton tunnels was judged necessary.

Edmonton tunnels are lined with a temporary lining consisting of rib and lagging and a permanent concrete lining. Because of the lack of field data, the loading was assumed to be uniform and was calculated according to Terzaghi's arching theory (1943). Drained soil parameters were used in the analysis for both linings, since the temporary lining might be the only support for as long as four months. In the long term, the temporary-lining was assumed to disappear by decay and the concrete lining was designed to withstand the full loading. The rib size and the concrete thickness were selected so that the maximum compressive stress in the lining does not exceed the maximum allowable compressive stress for the steel or concrete. The maximum thrusts were calculated simply by dividing the total vertical load on the tunnel above the springline by the cross-sectional area of the liner.

The previous assumptions led to approximations of the actual stresses on the safe-side but by an unknown amount. With the availability of more powerful computational facilities, a more refined analysis could be undertaken to approximate more closely the actual conditions: the vertical loading is not uniform, and the horizontal loading varies with depth depending upon the local geologic conditions. The load, due to undrained conditions, is first fully supported by the temporary lining; with time, drained conditions prevail and the load is modified. With sufficient time, the stress redistribution within the soil mass leads to a new static equilibrium and no further deformations occur, with the exception of possible creep effects. If the permanent lining is set up at this stage, no load is applied upon it. If the permanent lining is set up while the temporary lining is still deforming, the loads are shared between the two linings and the deformations come to a stop under the resistant forces developed in both linings. The time effects are two-fold: creep movements might develop and modify the loading, and the lagging might decay with time; however, no precise information is available on these effects.

On the basis of the previous description, it can be understood that the actual load distribution at a certain time is still controversial, due to the lack of field data. In order to provide for any findings from a precise measurement program, the calculations must be performed with a method which can be adopted to any type of load distribution. The purpose of the study is not to develop a new method of analysis but to apply a reliable method to the particular case of Edmonton. This led to the selection of a method which presents the required flexibility and which has been used sufficiently to be considered reliable. After reviewing the available methods of analysis (Chapter 2), the method of Anders Bull was chosen. It was first described in 1946 (Anders Bull, 1946) and has been widely applied since then. The lining is approximated by a polygon and the soil is assumed to be elastic. The method is described in detail in Chapter 3 and an example of its application to the Garrison Dam Tunnels is described in Chapter 4.

The method has the potential of predicting moments, thrusts and deflections of the lining as soon as a fairly accurate load distribution has been measured. It is of interest to study the differences due to several types of assumed loadings and to study the relative importance of soil parameters needed in the analysis (Chapter 7).

The study which follows provides a practical method of calculating moments, thrusts and deflections of a lining under a given symmetric loading. It contributes also to the improvement of the design approach by showing the important factors which govern the lining behaviour.

Chapter II

Literature Review

Many theories are available for the calculation of the stresses in a lining. These theories can be classified into two categories: the lining can be considered alone and subjected to various empirical or semi-empirical loadings, or the medium alone is considered, and the effect of a supporting liner is added by various approximations.

The first group of theories require knowledge of the load distribution on the lining. The first models used uniform loads acting vertically and horizontally. Hewett-Johannesson (1922) proposed a loading which took into account the earth pressure as a function of depth, the horizontal pressure as a function of the K_0 value, and the hydrostatic water pressure. The arching effect was provided for by taking only a fraction of the applied stresses under full overburden pressure. Terzaghi (1943) proposed a more systematic study of the arching [illegible] mechanism, while Protodyakonov (quoted in Szechy - 1967) proposed [illegible] calculation of the roof pressure as the weight of a parabola for [illegible] dimensions are given. In these early calculations, the [illegible] assumed to be perfectly rigid and the applied loads to be [illegible]. Such assumptions led to very conservative designs.

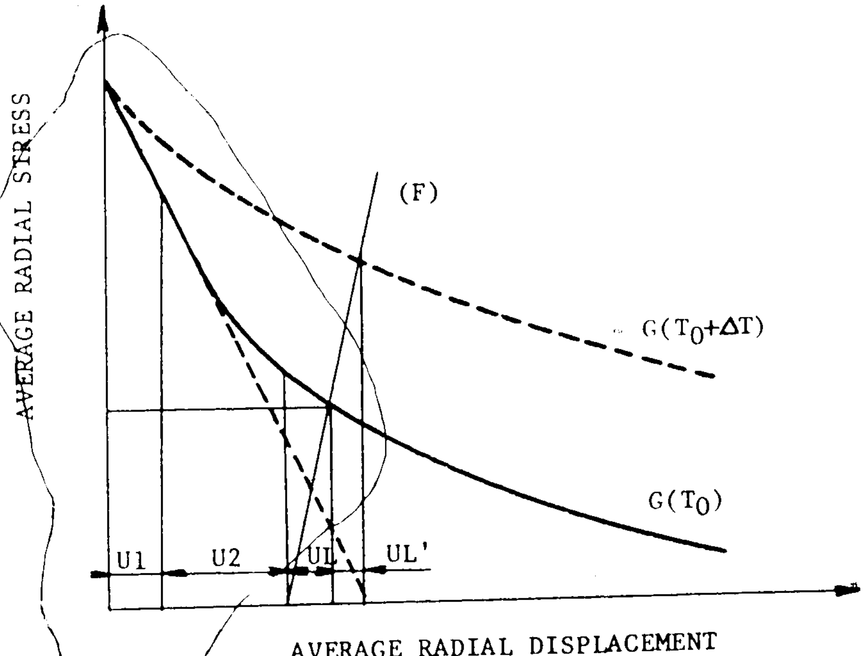
With a flexible lining, the tunnel is allowed to deform and to generate soil reactions which tend to decrease the moments and consequently the thickness of the lining. The soil is assumed to stay within the elastic domain and it is characterized by a soil reaction coefficient which is the force required to push a 1 sq. ft. rigid plate 1 inch into the ground. With this description of the soil, Bodrov-Gorelik (quoted in Szechy - 1967) proposed a mathematical analysis of the problem giving moments, thrusts and deflections under a given loading. Anders Bull (1946) and later Wissman (1968) proposed approximating the lining by a polygon with each segment elastically supported by a spring to approximate the elastic behaviour of the soil. Thrusts, moments and deflections are calculated by use of tabulated coefficients. The limitations of these methods are that the soil reaction coefficient is assumed to be constant throughout the soil and only a linear elastic behaviour is investigated.

Elastic and plastic soils have been studied mathematically by R. Fenner (1938) and elasto-plastic soils by S. Reyes (1966) applying the Finite Element Method to unlined openings. Brown, Green and Ramsey (1968) extended the application of the Finite Element Method to the design of culverts.

According to Deere and Peck (1969) the effects of construction procedure may totally invalidate the conclusions of the previous methods. The soil deforms by moving towards the working face even before the mole actually reaches it, deforms further until a lining is installed, continues to deform with the lining until the stress redistribution around the tunnel leads to a new static equilibrium; with time, further deformations may develop. Each deflection implies a reduction in the loads applied on the lining, according to an empirical or semi-empirical curve which is shown in Fig. 2.1. If an estimate of the deformations at each construction stage is available, the corresponding load is immediately known. It must be noticed that the ground reaction curve may vary with time.

Once such a ground reaction curve is known, it can be approximated by straight lines and each of these segments can be considered as a portion of an elastic soil. The methods of Bodrov-Gorelik and Anders Bull can be readily adapted to the problem.

In this thesis, primary consideration was given to the development of a convenient tool for use in the design of tunnel linings. The method of Anders Bull (1946) yields directly moments, thrusts, and deflections of the lining under a given loading. Its application was restricted to symmetrical loadings, although a program can be written for unsymmetrical load distributions. The method has the advantages of presenting a great flexibility to apply to various evaluations of the original stress distribution in the



U1, SOIL DEFORMATION BEFORE THE EXCAVATION REACHES THE POINT.

U2, SOIL DEFORMATION OF THE UNLINED TUNNEL.

UL, SOIL DEFORMATION OF THE LINED TUNNEL.

UL', SOIL DEFORMATION DUE TO CREEP.

FIG.2.1. - GROUND REACTION CURVE (G) AND SUPPORT REACTION CURVE (F)

ground. It also applies to any type of material, except ideally plastic, provided a ground reaction curve is available and can be reasonably approximated by straight lines.

Chapter III

Anders Bull Method for Computing Thrusts, Moments, and Deflection of a Lining

3.1 Description of the method.

3.1.1 Basic principles.

The method presented in this chapter is due to Anders Bull (1946). The tunnel lining deforms under a given earth pressure distribution and generates soil reactions at the points where the lining pushes outwards. The surrounding soil is assumed to remain in the elastic domain and the soil reactions are proportional to the displacements. This leads to the approximation of the tunnel lining by sixteen segments which are radially supported by springs to simulate the effect of the elastic soil reactions. Concentrated loads acting at the mid-point of each segment are then computed to approximate the earth pressure distribution.

3.1.2 Theoretical analysis of the problem.

- Thrust, moment, deflection at invert:

In a first phase, the thrust, the moment and the deflection at the tunnel invert are calculated for a loading consisting of a single concentrated force P. (Fig. 3.1). The deflection at the invert may be written as:

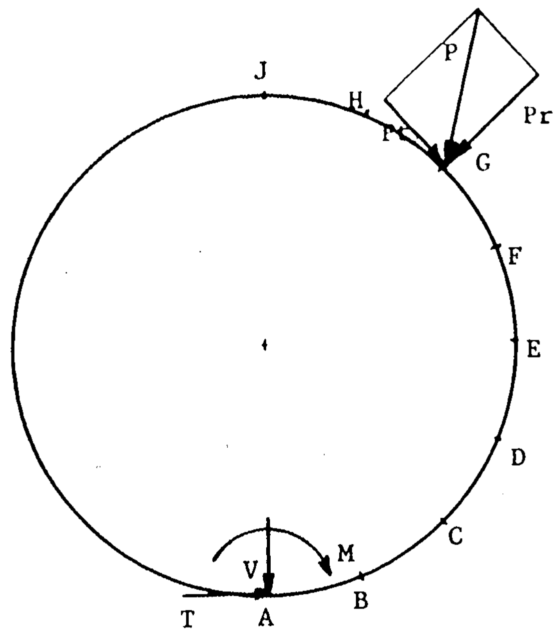


FIG. 3.1 - CALCULATION OF THRUST, MOMENT, DEFLECTION AT JOINT A UNDER THE ACTION OF A UNIQUE EXTERNAL FORCE P.

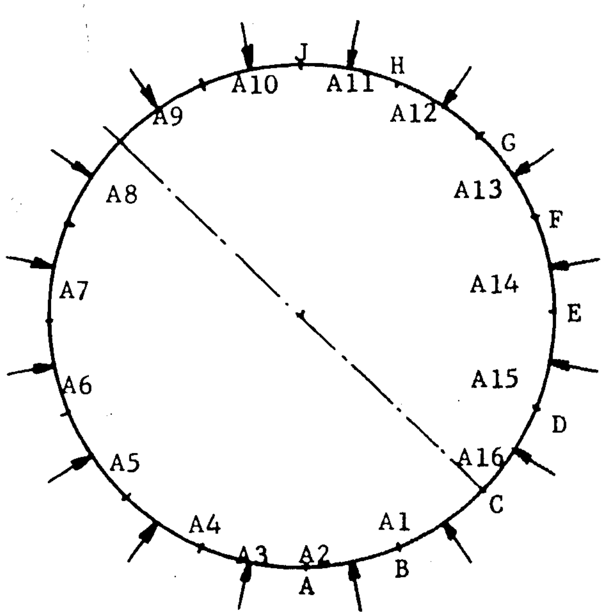


FIG.3.2 - SHIFTING OF A CONSTANTS FOR CALCULATION OF THRUST, MOMENT, DEFLECTION, AT JOINT C.

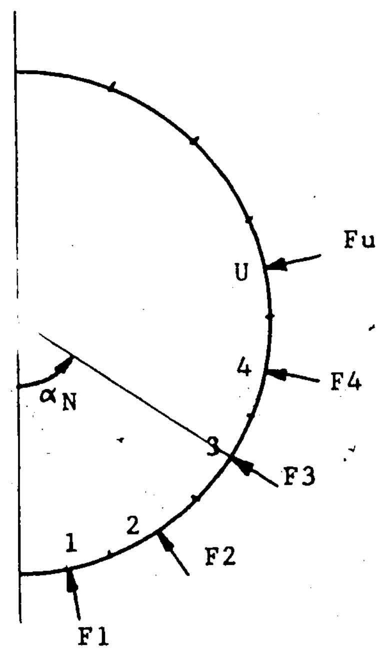


FIG.3.3 - SYMMETRICAL LOADING, AND SOIL REACTIONS DEVELOPING AT THE U LOWER POINTS.

$$d = \frac{R^3}{EI} \times f(T,V,M,PR,PT)$$

where f is a linear function of the thrust T, the moment M, the shear force V, the radial load PR and the tangential load PT. Since the quantities T,V,M are linearly dependent on PR and PT, the deflection d is also linearly dependent on PR and PT and the principle of superposition is applicable to d,T,V,M when more than one external force is applied to the lining.

$$d = \frac{R^3}{EI} \left(\sum P_R C_R + \sum P_T C_T \right)$$

$$T = \sum P_R A_{TR} + \sum P_T A_{TT}$$

$$V = \sum P_R A_{VR} + \sum P_T A_{VT}$$

$$M = R\left(\sum P_R A_{MR} + \sum P_T A_{MT}\right)$$

The literal expressions of the coefficients A_T, A_V, A_M are given in Appendix I.

- Thrust, moment, deflection at any joint.

To calculate d, T,V,M at any other joint, the figure is simply rotated until the joint under study is again between the joints where coefficients A_1 and A_{16} are applied (Fig. 3.2). The coefficients A are attributed to the joints now facing them. Because of the sign conventions in the calculation of the A coefficients, their sign must be modified.

A correction for the coefficients used in the calculation of moments is also included to take into account the approximation of the tunnel lining by a polygon. (Appendix I)

$$- M = - 0.0164 \frac{P'_R + P''_R}{2}$$

P'_R and P''_R being the radial loads applied to the segments adjacent to the joint under study.

In the case of a symmetrical loading, the coefficients A relative to symmetrical joints are added and the correction for the moments is applied. The new coefficients are referred to as B coefficients and the coefficients C relative to the deflections lead to the new coefficients D. Applying the principle of superposition, the deflection, the moment and the thrust at point N may be written for a symmetrical loading as:

$$d_N = \frac{R^3}{EI} \left(\sum P_R D_R (N) + \sum P_T D_T (N) \right)$$

$$M_N = R \left(\sum P_R B_{MR} (N) + \sum P_T B_{MT} (N) \right)$$

$$T_N = \sum P_R \, B_{TR} \, (N) + \sum P_T \, B_{TT} \, (N)$$

with B_{MR} and B_{MT} the moment and thrust coefficients to be applied to radial loads, and B_{MT}, B_{TT} the moment and thrust coefficients to be applied to tangential loads.

- Assumptions concerning soil reactions and tangential loads.

The soil reactions are assumed to be proportional to the displacements of the joints.

$$F = \rho \, K \, A'$$

F soil reaction (kips)

ρ displacement (in.)

K soil reaction coefficient

A' area of lining on which F is acting.

The soil reaction coefficient K is the force in kips which is required to push a surface of lining of 1 sq. ft. 1 inch into the soil. Throughout the following calculations, it is assumed that the width of the tunnel ring is 1 ft.

A mixture of gravel and grout is usually injected behind the concrete lining to ensure a good contact between lining and soil. For a day or two, this mixture is more or less in a plastic state and cannot resist shear forces. Later on, the shearing forces between soil and ring surface are replaced by tangential compressive forces in the adjacent ground. When considering a rib and lagging support, the boards will slide against the ribs under shear action, but without transmitting by friction any consequent tangential forces to the lining. When the permanent concrete lining is cast in place, there will not be either transmission of shear forces from the ground action, because of interference of the rib and lagging lining. Hence the tangential forces acting on the lining will be considered as due only to the weight of the lining.

3.1.3 - Calculation of thrust, moment deflection and soil reaction.

Assume that the loading is symmetrical and that the soil reactions will develop at the u lower points of Figure 3.3. For each of these points the soil reaction is proportional to the radial deformation:

(1) $F_N = (\delta \cos \alpha_N + d_N) K A'$

with δ settlement of the tunnel invert

$\delta \cos \alpha_N$, radial settlement at point N

d_N, radial deflection at point N

K, soil reaction coefficient

A', area of the 1 ft. wide segment projected on the plane of its chord.

The deflection at these points is due to the combined action of the radial forces P_R, the tangential forces P_T and the soil reactions F:

$$(2) \quad d_N = \frac{R^3}{EI} \left[\sum_1^8 P_R D_{RN} + \sum_1^8 P_T D_{TN} + \sum_1^u F D_{RN} \right]$$

Eliminating d_N between equations (1) and (2) yields u equations with u + 1 unknowns, i.e., δ, F_1, F_2, ..., F_u. Assume that the applied forces are in equilibrium and project them on the vertical axis:

$$(3) \quad \sum_1^u F \cos \alpha + \sum_1^8 P_R \cos \alpha + \sum_1^8 P_T \sin \alpha = 0$$

With the u previous equations there is now available a system of u + 1 linear equations with u + 1 unknowns. When solving the system, we find δ and the u soil reactions, hence the deflections from equation (2) and the moments and the thrusts from the following equations:

$$M_N = R\left(\sum_1^8 P_R\, B_{MR}(N) + \sum_1^u F\, B_{MR}(N) + \sum_1^8 P_R\, B_{MT}(N)\right)$$

$$T_N = \sum_1^8 P_R\, B_{TR}(N) + \sum_1^u F\, B_{TR}(N) + \sum_1^8 P_T\, B_{TT}(N)$$

Thrusts, moments and deflections are now known, but with the assumption that u selected points are moving outwards. The calculated deflections must be consistent with this assumption. Otherwise, another set of points moving outwards must be chosen until the assumption is satisfied.

3.2 Description and use of the computer program:

The computer program has been written to calculate thrusts, moments, deflections, soil reactions relative to a tunnel lining of any diameter in any type of elastic material, under a symmetrical loading. The calculations are based upon the method of Anders Bull and the basic assumptions concerning the loading are respected, i.e, the only tangential forces acting on the lining are due to the weight of the lining itself.

Type of lining

A non-reinforced homogeneous concrete lining and a rib and lagging lining have been considered. However, the program can be extended without any modification to any type of lining, composite ones or steel liner plates. A code differentiates the two cases which have been studied, since a spacing value must be given only when ribs are used. KODE equal to 1 corresponds to a rib and lagging lining while KODE equal to 0 corresponds to a concrete lining.

Units:

The data must be input with the units given in Table 3.1 because certain coefficients within the program have been computed according to this particular set of units.

Sign convention:

The radial loads, the radial stresses and the horizontal stresses due to earth pressure are counted positively when they act on the lining towards the center of the tunnel. The tangential loads are counted negatively when they act clockwise. Tensile stresses in the lining are counted positively and compressive stresses negatively, while moments which tend to decrease the radius of curvature of the lining are counted positively. The thrusts are always given positively. The radial displacements are positive at points where the tunnel deforms outwardly and are negative at points where the tunnel deforms inwardly.

Loading:

The loading may be defined by the radial and tangential forces applied on the lining at the midpoint of the lining segments or by the average horizontal and vertical stresses acting on each segment, or by use of a subroutine LOADNG included in the program which takes into account the soil properties, the tunnel geometry,the properties of the lining, the level of the ground water table, and which evaluates the loading for the full overburden pressure.

Arching effect:

If the loading is imposed, the arching effect must be taken into account in the calculation of the data. If the subroutine LOADNG is used, the arching effect must be simulated by taking as depth of the tunnel only a fraction of the actual depth.

Geometric properties:

For a concrete lining, the moment of inertia, the net section modulus, the weight of lining per foot of perimeter must be calculated for a ring 1 ft. wide.

For a rib and lagging lining or for steel liner plates, these quantities must correspond to the cross-sectional properties of the rib or of the liner plate.

Spacing:

The spacing must be equal to 1 ft. for a concrete lining, or equal to the width of the steel liner plates, or equal to the rib spacing, depending upon the type of support which is used.

Imposed original loading:

The loading may be defined by the horizontal and vertical stresses acting on each segment; these stresses must be calculated as the average stresses acting on a lining ring 1 ft. wide, regardless of the type of lining, because the modification of the loading according to the spacing is included into the program. These stresses must include the effect of the ground water table but not those due to the weight of the lining, since this is also provided for within the program. It should be noted that the vertical stresses at the points below the springline are due only to the head of water above them.

If the loading is defined by the radial and tangential forces, these values must be calculated for a 1 ft. wide ring of concrete lining or for a ring with a width equal to the rib spacing or to the width of the liner plates.

Input data:

The specific details about reading order are given in Appendix II for each option of loading.

The data file consists of one part of 5 to 8 lines which define the type of loading, the soil characteristics, the lining properties, the tunnel geometry, the number of segments on which soil reactions are assumed to develop, and another part which is a listing of the coefficients D (deflection), B_T (thrusts), B_M (moments), calculated by Anders Bull relative to radial and tangential loads. An example of a data file is given in Appendix II. It must be noted that the second part must always appear in the data file after the last line of the loading data.

Organization of the program:

The program is divided into a part which reads the data, part which evaluates the radial and tangential loads acting on the lining, a part which calculates thrusts, moments, deflections, soil reactions and a part which yields tables of results and plots of curves. The organisational chart (Figure 3.4) shows the general pattern of the program with the input and output files. A listing has been included in Appendix II.

Maximum stresses:

The maximum stresses in a lining cross-section occur on the inner and outer diameters. They have been calculated according to the formula:

$$\sigma_{maxi} = - \frac{T}{A} \pm \frac{M}{S}$$

which is valid provided that cracks in the liner do not develop.

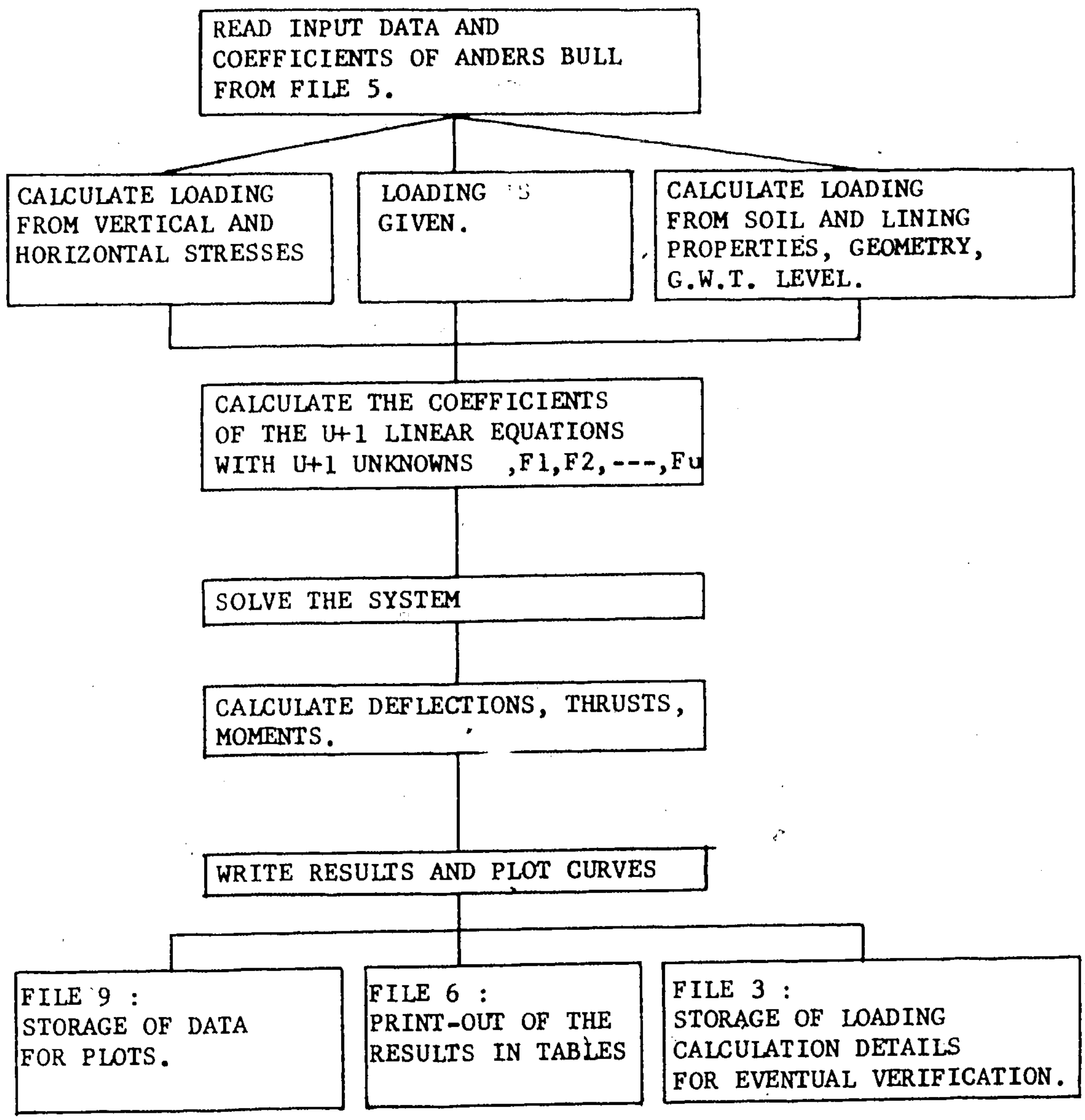


FIG.3.4 - ORGANISATION OF THE PROGRAM

A minus sign indicates compression. T is the thrust, M the moment, A the cross-sectional area, S the section modulus.

Output data:

A summary of the input data is first presented to allow easy verification, along with the loading as calculated in the program. The soil reactions, the deflections, the thrusts, the moments, and the inner and outer stresses are tabulated with the units given in Table 3.1. The output data can be plotted automatically if desired. A complete example is given in Appendix II.

3.3 - Check of the program:

The subroutine "loading" has been repeatedly checked by hand computations and compatibility of the three types of loading has been verified satisfactorily.

The calculation of thrusts, moments, deflections, have been checked on an example published by Anders Bull in the original paper. The soil and lining properties were as follows:

RO,	outside radius	8.71 ft.
R,	neutral radius	102.1 in.
A,	lining section area	27.6 in^2
AI,	moment of inertia	136. in^4

TABLE 3.1 - SUMMARY OF SYMBOLS AND UNITS.

INPUT DATA :

KODLD	CODE INDICATING THE SELECTED LOADING OPTION	
B	WIDTH OF THE RING FOR WHICH PR,PT, V, H, ARE CALCULATED	FT.
PR(I)	RADIAL LOAD ON LINING AT POINT I	KIPS
PT(I)	TANGENTIAL LOAD ON LINING AT POINT I	KIPS
V(I)	VERTICAL STRESS ACTING ON LINING AT POINT I	KSI.
H(I)	HORIZONTAL STRESS ACTING ON LINING AT POINT I	KSI.
KODE	CODE EQUAL TO 0 FOR STEEL RIBS AND 1 FOR CONCRETE	
GMA	UNIT WEIGHT OF SOIL	PCF.
GMS	SUBMERGED WEIGHT OF SOIL	PCF.
GMW	UNIT WEIGHT OF WATER	PCF.
SK	COEFF. OF EARTH PRESSURE AT REST	
DEPTH	DISTANCE FROM TUNNEL CROWN TO EARTH SURFACE	FT.
HW	DISTANCE FROM GROUND WATER TABLE TO SURFACE	FT.
PW	WEIGHT OF LINING PER FOOT OF PERIMETER, WIDTH B	LBS./FT.
RO	TUNNEL OUTSIDE DIAMETER	FT.
SPACNG	RIB SPACING	FT.
N	NUMBER OF SEGMENTS ON WHICH SOIL REACTION DEVELOP	
R	LINING MEAN DIAMETER	IN.
AK	SOIL REACTION COEFFICIENT	KIPS.
AI	MOMENT OF INERTIA OF THE LINING, XX AXIS	IN^4
E	YOUNG'S MODULUS OF THE LINING	KSI.
AS	SECTION MODULUS OF THE LINING	IN^3
A	CROSS-SECTIONAL AREA OF THE LINING	IN^2
NZ(I)	POINTS ON WHICH SOIL REACTIONS DEVELOP	

OUTPUT DATA :

B(I)	SOIL REACTIONS	KIPS
AD(I)	RADIAL DEFLECTIONS OF LINING	FT.
AT(I)	THRUSTS ON THE LINING	KIPS
AM(I)	MOMENTS AT JOINTS I	KIPSxFT.
SIGO(I)	STRESS AT JOINT I ON THE OUTER SIDE OF THE LINING	KSI.
SIGI(I)	STRESS AT JOINT I ON THE INNER SIDE OF THE LINING	KSI.

TABLE 3.2 - COMPARISON OF THE RESULTS OF THE PROGRAM ON A NUMERICAL EXAMPLE GIVEN BY ANDERS BULL.

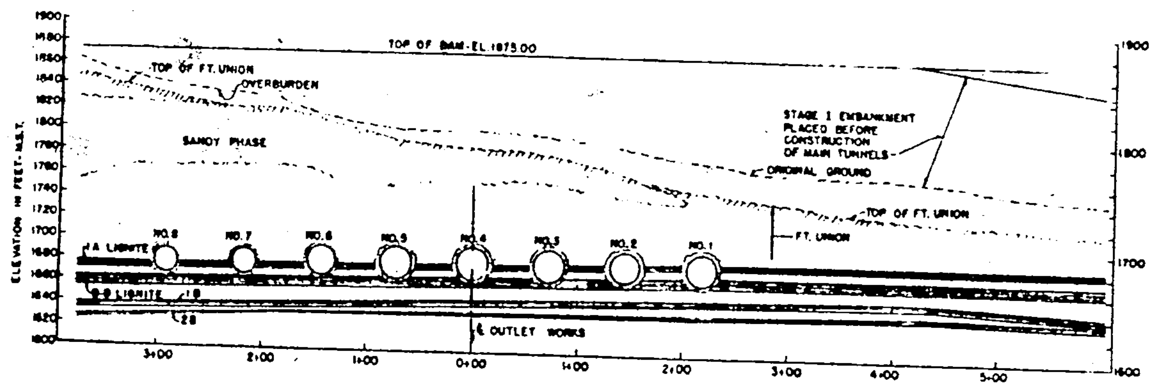
POINT	RADIAL FORCES(KIPS)	TANG. FORCES (KIPS)
1	8.966	-.147
2	10.388	-.418
3	12.144	-.625
4	12.920	-.738
5	12.505	-.738
6	13.102	-.625
7	14.290	-.418
8	15.159	-.147

WITH THIS LOADING DATA FROM ANDERS BULL AND THE DATA GIVEN IN THE TEXT, THE FOLLOWING RESULTS HAVE BEEN OBTAINED :

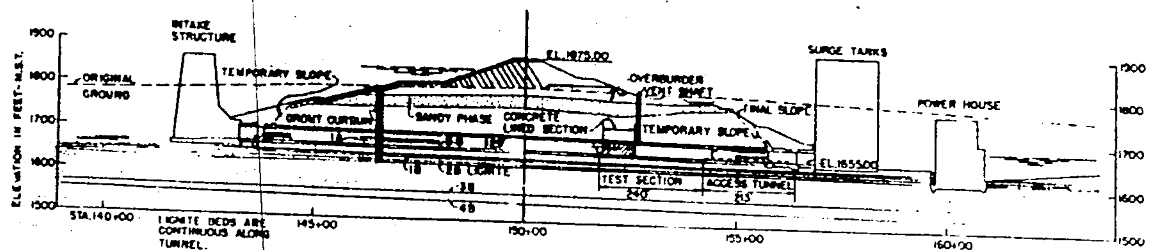
	RESULTS FROM ANDERS BULL	RESULTS FROM THE PROGRAM	JOINT OR POINT
DEFLECTIONS (IN.)	.1508	.1512	1
	.1264	.1260	2
	.0947	.0948	3
	.0673	.0672	4
	.0284	.0288	5
	-.0540	-.0540	6
	-.1678	-.1680	7
	-.2524	-.2520	8
MOMENTS (IN.xKIPS)	5.31	5.4	A
	-4.5	-4.3	B
	-20.5	-17.6	C
	-15.3	-15.4	D
	+22.6	+22.7	E
	+51.2	+51.4	F
	+24.4	+24.1	G
	-31.7	-31.8	H
	-58.2	-58.2	J
THRUSTS (KIPS)	37.704	37.721	F
	35.486	35.483	J
MAX.COMPRESSIVE STRESS (KSI)	-3.56	-3.56	F
MAX. TENSILE STRESS (KSI)	1.20	1.20	J

AS,	net section modulus	23.4 in^3
AK,	soil reaction coefficient	12.

The radial and tangential loads computed by Anders Bull were input in the program and the results, summarized in Table 3.2, are found identical, when it is considered that the slight differences can be attributed to the superior precision of the computer, especially in the delicate evaluation of the moments.



SECTION THRU ℄ DAM
LOOKING UPSTREAM



PROFILE-TUNNEL NO.4

FIG.4.1.- PROFILE OF THE GARRISON DAM TUNNELS - (BURKE - 1957)

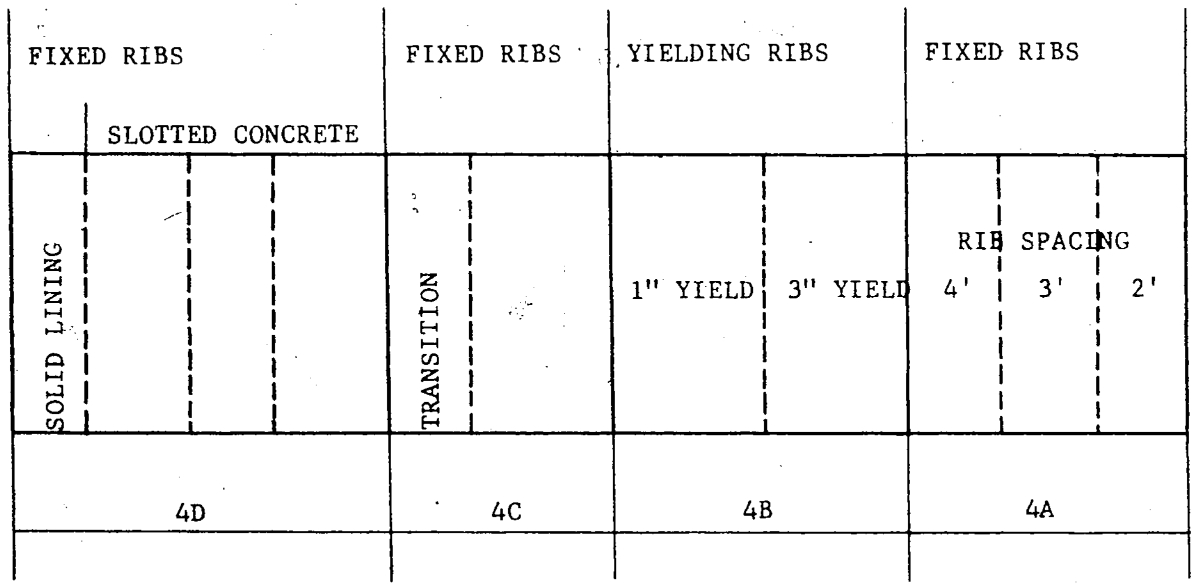


FIG.4.2 - ARRANGEMENT OF TEST SECTION - (BURKE - 1957)

similar to the temporary support used in Edmonton.

Once the first results were available from the tunnel test section, the design of the main tunnels was modified and the actual work started. Measuring devices were also installed in the main tunnels. In order to provide a valid comparison of the results, the tunnel sections have been classified according to the type of loading they were subjected to. The tunnel test section loading has been labeled "single tunnel case", since there is no interference with an other tunnel being built nearby. Sections 5E, 7E, 2E, 2B, 2F of the main tunnels belong to this category. Furthermore, the loading must be similar, i.e., in the case of a rib and lagging lining, the results for the main tunnels are considered only for the first three months, prior to the placement of the concrete lining. Table 4.1 summarizes the measurements for several sections during the temporary support period.

Burke and Lane (1957) assumed that the loading could be approximated by a uniform vertical pressure acting on the upper part of the tunnel. They calculated a measured load based on the collected data, while developing a relative yield method to predict such a loading. The results appear in Table 4.2.

The first check will be to determine whether the assumption of a uniform vertical pressure is justified or not. Three different loading patterns, including the one proposed by Lane, are considered and the thrusts are calculated by the method of

TABLE 4.1 - SUMMARY OF MEASUREMENTS (BURKE,1957)

STEEL RIBS DURING TEMPORARY SUPPORT PERIOD FOR SINGLE TUNNEL CASES.								
SECTION		2B	2F	2E	7E	5E	4A	4C
RIB SPACING (FT.)		4.	4.	3.5	4.	3.	3.	3.
BORE DIAMETER (FT.)		35.	35.	35.	27.	35.	36.	36.
RIB SIZE		8WF48	8WF48	10WF72	10WF72	10WF72	12WF99	12WF99
	CLOCK							
MAX.WEB STRESS	3	6,600	6,000	10,400	3,300	8,300	4,070	2,160
COMPRESSION	6	7,200	4,600	10,400	2,200	9,700	3,820	2,250
(PSI.)	9	12,400	5,900	16,100	5,300	8,600	3,300	2,160
	12	6,800	9,000	12,500	3,600	8,300	3,980	2,160
MAX.THRUST	3	23.3	21.2	62.9	17.5	58.6	39.5	21.0
KIPS/L.FT.	6	25.4	16.2	62.9	11.7	68.5	37.1	21.8
	9	43.8	20.8	97.4	28.1	60.7	32.0	21.0
	12	24.0	31.8	75.6	19.1	58.6	38.6	21.0
MAX.MOMENT	3	-60.	-110.	-100.	+80.	-70.	+191.	+153.
(IN.xKIPS/L.FT.)	6	-170.	-50.	-90.	+80.	+180.	-175.	-197.
	9	-120.	+280.	+780.	-60.	-320.	+155.	+225.
	12	+190.	+260.	+360.	+200.	+300.	+292.	+350.
MAX.OUTER FIBER		40,000	2,600	39,100	-	-	8,700	20,000
STRESS (PSI.)	CLOCK	6	6	6	12	6	6	3
MAX.HORIZONTAL		+0.3	+0.2	+0.7	+0.1	+0.4	+0.9	+1.1
DIAMETER CHANGE (IN.)								
MAX.VERTICAL		-0.2	-0.2	-1.1	-0.3	-0.4	-1.1	-1.4
DIAMETER CHANGE (IN.)								

+ MEANS THE APPLIED MOMENT INCREASES RADIUS OF THE RIB.

+ MEANS TENSILE STRESS IN THE RIB.

TABLE 4.2 - LOADING EVALUATION FOR THE SINGLE TUNNEL CASE.
(LANE, 1957)

SECTION OF TUNNEL	OVERBURDEN LOAD (TSF.)	LOAD COMPUTED BY RELATIVE YIELD METHOD (TSF.)	LOAD BACKCALCULATED BY LANE (1957) (TSF.)
4A	6.6	1.1	1.0
4C	6.6	0.55	0.60
5E	10.7	1.8	1.60
7E	10.9	1.4	0.35
2E	10.8	1.4	1.55
2B	5.9	0.5	0.60
2F	5.7	0.5	0.45

Anders Bull and compared to the measurements.

4.2 Selection of a pattern of earth pressure distribution:

The thrusts are the comparison criterion for the following study. The only available data consists of the measured thrusts, moments and deflections. The problem is to determine the pattern of load distribution which leads to the observed thrusts and moments in a given section. When properly designed several loading patterns can lead to the measured thrust distribution, but the proper loading pattern leads to results which are consistent with the observed moments and deformation distributions. This study is restricted to three types of loading on a given section.

Measurements in the tunnel test section have been carried out on more ribs than in the sections of the main tunnels. Hence, the results are more reliable and Section 4A of the test section is chosen for the study. The three different loading patterns which are compared are shown in Table 4.3. Loading (1) has been proposed by Lane (1957) and is based upon interpretation of the measured data. The uniform vertical load is 1 tsf., and the horizontal load is null. Loadings (2) and (3) are both hypothetical loadings which call for a hydrostatic distribution of the vertical pressure. The horizontal load is null in loading (2) and is calculated according to the K_0 value in loading (3). Using the method of Anders Bull, the overburden height H has been chosen for each of these loadings as the height for which the calculated thrusts are the closer to the measured

TABLE 4.3 - SELECTION OF A LOADING PATTERN.

THE RESULTS OBTAINED BY THE METHOD OF ANDERS BULL FROM LOADINGS (1),(2),(3), ARE COMPARED TO THE MEASURED VALUES TO DETERMINE THE MOST PROBABLE LOADING ON SECTION 4A.

	CLOCK	MEASURED DATA	LOADING (1)	LOADING (2)	LOADING (3)
THRUSTS	6	111.3	108.35	111.41	114.95
(KIPS)	3,9	96.0,118.5	106.63	109.69	113.04
	12	115.8	103.87	107.21	111.45
MOMENTS	6	+43.6	-.4846	-.4990	+0.0149
(FT.xKIPS)	3,9	-38.8,-47.7	-.0030	-.0816	-2.9021
	12	-73.1	-19.0305	-13.6541	0.2485
MAX.OUTER FIBER STRESS		+8,700	-5,260	-4,900	-4.49
(PSI.)	CLOCK	6	12	12	4.5
HORIZONTAL DIAMETER CHANGES (FT.)		.0750	.0014	.0014	-.0020
VERTICAL DIAMETER CHANGES (FT.)		-.0920	-.0035	-.0031	+.0011

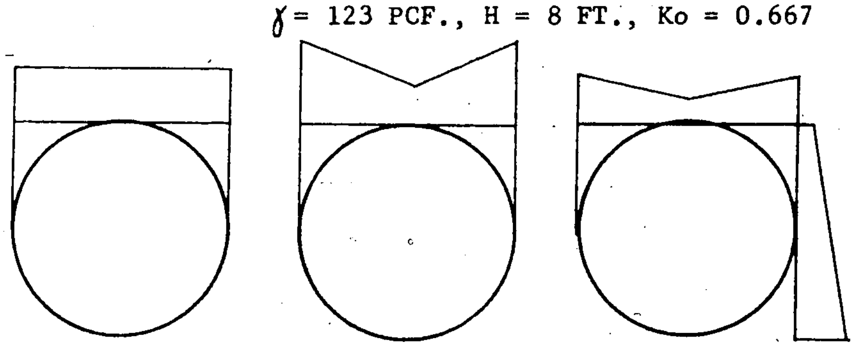
WITH LOADING (1) : VERTICAL STRESS = 1TSF., HOR.STRESS = 0.TSF.

LOADING (2) : VERTICAL STRESS = γ xH , HOR.STRESS = 0.TSF.

γ = 123 PCF., H = 17.5 FT.

LOADING (3) : VERTICAL STRESS = xH , HOR.STRESS = Ko x σ_v

γ = 123 PCF., H = 8 FT., Ko = 0.667



LOADING (1) LOADING (2) LOADING (3)

thrusts. The values of the height H were found to be 17.5 ft., for loading (2) and 8 ft., for loading (3) with a K_0 value of 0.667. Loadings (1), (2) and (3) differ from the total overburden pressure because of the arching effect in the overlaying soil.

The three loading patterns are compared in Table 4.3. The moments, thrusts and deflections have been obtained by use of the method of Anders Bull. Loadings (1) and (2), characterized by no horizontal pressure, yield similar results in agreement with the general deformation pattern of the lining. Loading (3) however indicates a shortening of the horizontal diameter with a lengthening of the vertical one, in opposition to the field results. Loading (3) will then be disregarded in further analysis of this case history. Loadings (1) and (2) give moments and deformations which, although correct in signs, are much lower than the measured values. This will be discussed in more detail in Section 4.4 - Table 4.3 indicates that moments and deformations should not be considered as quantitative elements to check the method. The differences are attributed to accidental irregularities in loading and in construction procedures, according to Burke (1957).

In summary, both loading patterns (1) and (2) yield results in good agreement with the observed data, but the loading pattern (1) will be preferred in further comparisons because of its greater simplicity.

4.3 Selection of an evaluation of the earth pressure:

In the previous section, a uniform vertical pressure combined with a null horizontal pressure has been shown to lead to a good approximation of the observed data. This loading pattern will then be used in all the following sections. Now that a loading pattern has been selected, the problem is to determine the uniform vertical pressure itself. In the case of the rib and lagging lining, the original data consists of rib stresses and deformations. From this data, it is possible to back calculate the applied uniform vertical pressure by various assumptions. Lane (1957) proposed values of the vertical pressure for each section but without making clear what data they were obtained from. He developed also a semiempirical method, which he refers to as the relative yield method, which leads also to an estimate of the uniform vertical pressure. Finally, the author of this thesis calculated the average uniform vertical pressure which was applied from the thrust data published by Burke (1957). For each section of the tunnel which has been instrumented, three evaluations of the loading pressure are now available. In order to determine the most reliable one, these evaluations are compared in Table 4.4 to the uniform vertical pressure which is found by trial and error by the method of Anders Bull and which best approximates the average measured thrust of each section. Table 4.4 shows that the loading

TABLE 4.4 - SELECTION OF A LOADING PRESSURE EVALUATION.

SECTION	2B	2F	2E	7E	5E	4A	4C
VERTICAL STRESS (KSF.)							
(1)	1.333	1.000	3.444	0.778	3.556	2.222	1.333
(2)	1.111	1.111	3.111	3.111	3.999	2.444	1.222
(3)	1.660	1.280	4.266	1.415	3.520	2.050	1.180
(4)	1.700	1.333	5.000	1.600	3.800	2.300	1.330

(1) : V.STRESS BACKCALCULATED BY LANE (1957) FROM MEASURED DATA.

(2) : V.STRESS CALCULATED BY LANE (1957) WITH THE RELATIVE YIELD METHOD.

(3) : V.STRESS CALCULATED BY AUTHOR FROM THRUST DATA PUBLISHED BY BURKE (1957).

(4) : V.STRESS CALCULATED BY AUTHOR WITH THE METHOD OF ANDERS BULL BY TRIAL AND ERROR TO BEST APPROXIMATE THE MEASURED THRUSTS.

recalculated from thrust measurements is less subject to abrupt variations than the loading back calculated by Lane (1957) or than the loading calculated by the relative yield method. It is then considered more reliable, although not always the closest to the value which, when input into the program developed in this thesis, yields the best approximation of the measured thrusts.

4.4 Comparison of measured and calculated thrusts:

The two previous sections determined the most probable loading distribution which occurred in the field. With this knowledge, it is now possible to check the accuracy of the method of Anders Bull. Table 4.5 compares the measured average thrust to the calculated average thrust obtained by the method of Anders Bull under the loading determined in Section 4.3. The method of Anders Bull is shown to approximate the actual thrusts within 12% error and to give thrusts which are always lower than the measured ones.

4.5 Comparison of measured and calculated moments and deformations.

The calculated moments and deformations differ by a considerable amount from those measured. Section 4A provides typical results under the loadings (1) and (2) as shown in Table 4.3. According to Burke (1957), the bending stresses account for approximately 75% of the total rib stresses. Such high values of the

TABLE 4.5 - COMPARISON OF MEASURED AND CALCULATED THRUSTS

THE VERTICAL STRESS HAS BEEN BACKCALCULATED BY THE AUTHOR FROM THRUST DATA PUBLISHED BY BURKE (1957). THE THRUSTS ARE CALCULATED BY THE AUTHOR WITH THE METHOD OF ANDERS BULL FOR SECTIONS UNDER LOADING PATTERN (1).

SECTION	2B	2F	2E	7E	5E	4A	4C
VERTICAL STRESS (KSF.)	1.660	1.280	4.266	1.415	3.520	2.050	1.180
AV. CALCULATED THRUST (KIPS)	107	82.9	230.	67.4	162.5	98.6	58.1
AV. MEASURED THRUST (KIPS)	116.5	90.	261.	76.4	185.	110.4	63.6
RATIO CALCULATED THRUST/ MEASURED THRUST (%)	92	92	88	88	88	89	91

bending stresses cannot be attributed to the relatively moderate difference between vertical loading and horizontal ground reaction. Burke attributes them mainly to construction procedures: erection stresses and blocking load irregularities, caused by variations in the size and spacing of the blocks and by the concentrated load created at each blocking point by wedging. The ribs were also found to be subjected to considerable cross-bending.

However, the crown moment was constant in sign and nearly always the largest moment experienced by the rib. This is in very good agreement with the sharp increase in moments obtained at the crown with loadings (1) and (2). (Table 4.3).

The excessive difference between measured and computed deformations is correlated to the difference between measured and calculated moment distribution. However, one cannot conclude that the method itself is safe for the evaluation of deformations in a hypothetic undisturbed ca

4.6 Concluding remarks:

A critical analysis of the data published by Burke and Lane (1957) has led to the adoption of a uniform vertical pressure with no horizontal pressure distribution as the most likely loading pattern. Lateral pressures develop subsequently from the ground reactions due to the lining deformations.

The actual loading is not symmetric and the initial moments in the ribs are not zero but are of considerable magnitude even before any loading is applied, because of unavoidable erection stresses, while the computation method assumes a symmetric loading and moments only due to loading.

However, the average measured thrust is not very sensitive to departures from the ideal case and it has been used as a criterion to check the method of Anders Bull. The average computed thrust is approximately 10% lower than that measured. (Table 4.5). The 10% difference may be attributed to the asymmetry of the actual loading and to approximations in the back calculation of the loading itself.

Moments and displacements, which depend upon the initial construction stresses, do not correspond to the measured values. However, the method evaluates moments and deformations qualitatively, indicating the joints where maxima are likely to be reached, in good agreement with the field data.

Within these limitations, the method of Anders Bull may be considered reliable for the approximation of the rib thrusts, providing the loading is properly evaluated. Moments and deformations depend mainly on the construction procedures. Field data and computed data are in reasonably good agreement as shown in Table 4.5.

CHAPTER V

Tunnel design and construction practice of the City of Edmonton

5.1 Geology:

The sedimentary deposits in the Edmonton area may be divided, from the surface down, into four main strata, according to Beaulieu (1972):

- Glacial Lake Edmonton sediments, consisting of silts and clays deposited in a postglacial lake. The thickness varies from 16 ft., to over 40 ft. The average composition is 15% sand, 35% silt and 50% clay sizes. The deposits vary from clayey near the surface to silty at base of deposit.

- Till - This unstratified glacial deposit underlies most of the other Pleistocene deposits in the area. The till contains pockets of water-bearing sand. Its average composition is 43% sand, 38% silt and 19% clay sizes.

- Saskatchewan Sands and Gravels of early Pleistocene Age, consist of well sorted, rounded quartz sands with minor silt and clay. They occur in the area as terrace and valley floor deposits. Their average composition is 91% sand, 3% silt and 6% clay sizes.

- The bedrock consists of interbedded claystone, siltstone and sandstone strata with numerous coal and bentonite seams. The bedrock surface resulted from a long period of erosion during the Tertiary. Its main feature in the Edmonton area is the preglacial Beverly Valley extending west-east under the northern part of the City. Table 5.1 indicates the engineering properties of the most common materials in the Edmonton area.

5.2 Operation procedures:

From past experience and tests of several methods, the Water and Sanitation Department of the City of Edmonton selected as the most economical solution to tunnelling, the use of moles whenever possible for excavation, ribs and lagging for primary lining and either a cast-in-place concrete lining or precast concrete pipes with grouting as a permanent lining.

Precast pipes consist of one large pipe inside which a partition has been built. A single pipe is then used as a storm sewer for the major section and as a sanitary sewer for the smaller section. This system allows the treatment of the sanitary water only instead of the total sanitary-storm water.

The sequence of operations is as follows:

- 1. Preliminary exploration is undertaken.
- 2. Vertical shafts are drilled every 5,000 ft.

PROPERTIES	BEDROCK	SASKATCHEWAN SANDS	TILL	LAKE EDMONTON SEDIMENTS
SHEAR STRENGTH	HIGH TO LOW	HIGH	MEDIUM-HIGH	LOW-MEDIUM
MOISTURE CONTENT	15%	VARIABLE	18%	25%
LIQUID LIMIT	VARIABLE	/	40	65
PLASTIC LIMIT	VARIABLE	/	15	25
DRY DENSITY	COMPACT	MEDIUM-DENSE	DENSE	MEDIUM
S.P.T.	100-200	200-300	25-200	15-25
COMPRESSIBILITY	LOW	LOW	LOW	LOW-MEDIUM

TABLE 5.1 - ENGINEERING PROPERTIES OF TILL (BEAULIEU - 1972)

- 3. Excavation is performed in both directions for 2,500 ft. The ribs and lagging are set up just behind the cutting wheel of the mole. This stage may last up to 6 months, depending upon subsoil conditions and tunnel diameter.

- 4. When an average of 2,500 ft., has been excavated, the concrete lining is placed, starting from the end of the tunnel and proceeding towards the original shaft. This stage may last up to 4 months.

5.3 Preliminary exploration:

The exploration program consists of a series of bore-holes drilled close to the tunnel axis with a spacing varying in the range 100-500 ft., depending upon the expected difficulties. Borehole data provides a qualitative description of the subsoil materials. No laboratory tests are performed and the ground water table is assumed to be at an average depth of 30 ft.

When sand bodies are found and when further exploration proves that they communicate with some important water-bearing sand layer, the tunnel direction is modified whenever possible. Although compressed air, chemical grouting and drainage programs have been considered at times, none of these methods have ever been actually applied in Edmonton.

5.4 Excavation:

5.4.1 Types of moles:

Most tunnels in Edmonton are excavated with moles that fall into two categories:

- 1. Moles protected by a shield : in poor ground conditions, the excavating process is safer; in large tunnels, the shield length reaches 18 ft., which makes it difficult to guide when a curve is required.

- 2. Unprotected moles : This type of machine is particularly well adapted to drill an important radius of curvature; primary lining is set up 8 ft., behind the cutting wheel, whereas when using a shield-protected mole, it is necessary to wait until the shield clears the way. However, unprotected moles may allow development of large cavities in the crown of the tunnel when hitting a sand or silt layer.

5.4.2 Jacking forward of the mole:

When lagging is placed outside the ribs, the mole is advanced by means of 4 jacks pushing perpendicularly on the lagging. The lateral displacements of the boards acted upon may reach 4 inches locally.

When lagging is placed in between the ribs, the machine is jacked forward by pushing on the lining in the direction of the tunnel axis. In this case, the lagging works in compression and the surrounding soil is not disturbed.

5.4.3 Tunnel diameter:

The cutting wheel of the machine has a nominal diameter that may be taken as the average diameter of the unsupported tunnel just after excavation. Immediate deformations change this value somewhat. However, the nominal diameter itself changes with the condition of the cutting teeth on the rim of the mole. The variation in diameter may reach 2 inches. The ribs are dimensioned so that a minimum average clearance between ribs and ground allows easy placing of the lagging. When the teeth are new, clearance can be excessive in some sections where the ground is exceptionally good; clearance of 1 inch sometimes subsist 500 ft., behind the working face, thus after a time interval which is sufficient for most of the deformations to take place. Cutting teeth are changed when placing of the lagging becomes difficult (no more clearance).

According to the University of Illinois Report on Tunnelling (1969), clearance between ground and support is of utmost importance for the determination of the actual loading acting on the lining. From the previous description, it follows that the load should be maximum when using worn cutting teeth.

5.4.4 Rate of advance of the mole:

Depending upon the subsoil conditions, the rate of advance varies in the range 5 to 20 ft./shift, i.e., 15 to 60 ft./day. A 2,500 ft., excavation would require 1.5 to 5.5 months for completion.

5.4.5 Unsupported section of the tunnel:

In the case of an unprotected mole, a maximum length of 8 ft., for an outside diameter of 8 ft., is excavated without support.

The rate of advance is such that the unsupported time for any point of the section does not exceed 3 hours. If the work has to be stopped, some kind of temporary support is systematically provided.

5.4.6 Time dependence of deformations:

Because of financial problems, some tunnels have been left for more than two years with the primary rib and lagging lining only to support them; three months after excavation, no deformations were observed and this state was maintained until placement of the concrete lining; unfortunately no precise record of the deformation is available.

A time-deformation curve may be obtained by combining the following data: settlements of the ribs are available along the axis of some tunnels; through the history of the excavations, it is possible to determine the elapsed time between installation of the rib and measurement of its vertical diameter. The initial diameter is assumed to be the nominal one. For useful comparisons, a geologic profile along the axis of the tunnel is needed; borehole reports are also available for this purpose. Direct measurement of settlement of a particular rib with time would be of great interest. A time-deformation curve would be a useful asset in determining whether or not the permanent concrete lining will be subjected to any additional earth pressure during its life-time. However, the data available in Edmonton does not allow such a study yet.

5.5 Preliminary lining:

5.5.1 Design of a rib:

The arching theory of Terzaghi is used to determine the vertical pressure on the lining. With a rib spacing varying usually between 4 and 5 ft., the total load acting on a rib can be calculated and the rib size selected with factor of safety of 1.5. The rib sizes commonly used are listed in Table 5.3.

5.5.2 Lagging methods:

The lagging consists of 2 x 8 in. or 2 x 10 in., rough spruce boards. Depending on the type of the mole, the lagging is placed

I.D. OF TUNNEL (FT.)	O.D. INJECTION PIPE (IN.)	MINIMUM CLEARANCE BETWEEN RIB AND STEEL FORM (IN.)
5 TO 7	4.	6.
7 TO 12	6.	8.
16.5	5.5 *	7.5

*10 INCHES DIAMETER PIPE FLATTENED TO 5.5 INCHES.

TABLE 5.2 - MINIMUM CLEARANCE BETWEEN RIB AND STEEL FORM.

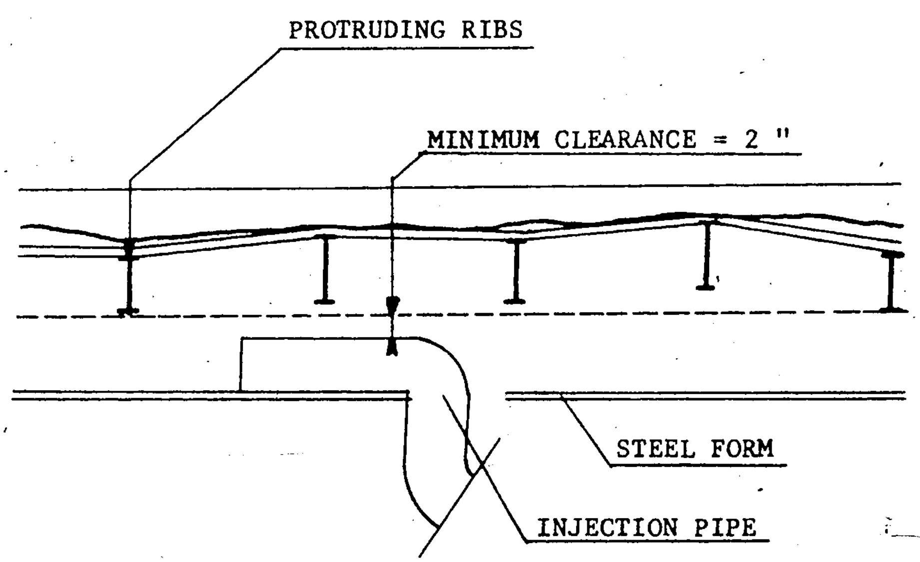


FIG.5.1 - TECHNICAL LIMITATIONS TO CONCRETE THICKNESS DECREASE.

outside or in between the ribs.

When the lagging is placed outside the ribs, the rib spacing can be readily modified in the field to accommodate unexpected conditions without having to cut the boards to a new length. In some sections, the lagging itself works in compression and supports part of the vertical pressure. The bulging which is induced has been observed to be so important that the lagging may be as far as 4 inches from the outside surface of the ribs at the springline. Because of this phenomenon and because of the irregularities due to the overlapping of 2 inch thick boards, the volume of concrete poured in place exceeds the volume calculated on the basis of an ideal lagging with a constant thickness of 2 inches.

When the lagging is placed in between the ribs, the rib spacing can be varied by increments only, and common lengths of lagging of 3, 4 or 5 ft., must be cut precisely. With serious subsoil conditions, the earth pressure is sometimes important enough to push the boards out of the steel beams, thus decreasing safety. This lagging method however,alleviates the problem of having ribs protruding into the concrete lining, which is of interest in case of an eventual reduction of the concrete thickness. The lagging in this configuration works longitudinally when the mole is jacked forward.

5.5.3 Varying the primary lining stiffness:

When poor subsoil conditions require departure from the recommended construction procedures, the lining stiffness is increased three ways:

- 1. Spacing is reduced according to the judgement of the tunnel surveyor or the foreman.

- 2. Wide flange ri[illegible]d instead of the cheaper I beams, which are more subject [illegible]re by deflection, but which can meet the average [illegible]ts for particular tunnels.

- 3. A better quality of steel is selected.

5.5.4 Sand or silt lenses:

When tunnelling through granular formations, cavities may develop in the roof of the tunnel. After placement of the primary lining, holes are drilled from the surface and the cavity is backfilled with sand. However, such a loading is not taken into account in the design of the ribs and it can result in failure of the ribs by buckling.

5.5.5 Decay of lagging:

This very controversial point implies the necessity or not of a concrete lining designed to withstand the total earth pressure.

A general agreement seems to exist that most of the strength of the lagging is conserved for the first fifteen years. It would be of considerable interest to study the strength of the lagging of the oldest tunnels in Edmonton. Samples are available when connections of a new tunnel with an old one are realized.

5.5.6 Clearance between lagging and ground:

To facilitate the placement of the lagging when the boards are placed outside the ribs, the intended clearance between lagging and ground is 1 inch. However, the actual clearance in firm ground varies from 0 to 3 inches because of overlapping.

When the poling boards are placed between the ribs, the clearance between lagging and ground depends only on the diameter of the base excavated by the mole. It varies from 0 to 2 inches.

5.6 Permanent lining

5.6.1 Construction practice:

The permanent concrete lining consists of precast pipes complete with grouting or of a cast-in-place lining. Thickness of concrete and of grouting have been compiled in Table 5.3. The operation procedures for the cast-in-place concrete lining are as follows:

- 1. The invert is poured for the whole length. When

TUNNEL LOCATION	DEPTH (FT.)	O.D.	STEEL	RIB	SPACING (FT.)	CONCRETE THICKNESS (IN.)	GROUT THICKNESS (IN.)
/	/	6'4"	A36	4I7.7	4.to 5.	8.	NONE
116St. and 102Av.North to 108Av.	/	8'3"	CORTEN B	4I7.7	5.	6.5	10. *
37Av.,113St.E. to 102St.	/	8'8"	CORTEN B	4I7.7	4. to 5.	10.	NONE
100Av.,170St.E. to 163St.	/	9'3"	A36	4x4WFe13	5.	7.5	9. *
30Av.,91St.W.to 99St.	/	11'11"	CORTEN B	4x4WFe13	5.	6.	10. *
163St.N. of 79Av. to 96Av.	60.	13'8.5"	CORTEN B	4x4WFe13	4.	13.5	NONE
79Av. and 156St.	/	16'6"	CORTEN B	6x6WFe25	5.	15.	NONE
45Av. and 106St.	/	18'0"	CORTEN B	6x6WFe25	4.	12.	NONE
30Av.from Whitemud Creek to Calgary Trail	115.	19'3/4"	CORTEN B	6x6WFe25	4.	12.	NONE

* Precast concrete pipes with inner partition.

Strength of concrete = 4000 psi. Strength of grouting = 100 psi. Lagging = 2"x8",2"x10"rough spruce.

TABLE 5.3 - EDMONTON TUNNEL DESIGN DATA.

completed, the railway track which is used for the removal of the debris is replaced and a steel arch form is set in place for the crown pour.

- 2. The steel arch form is divided into collapsable sections 10 to 25 ft., long. The length of the total form represents the concrete pour per day. Concrete is dumped from the surface into a pipe installed into the original shaft or into smaller holes separated from the working area by a maximum of 1,000 ft. This distance represents the limit capacity of the concrete pump which supplies the working area. A compressor is used for additional power for pouring. Concrete is injected behind the form through a pipe placed between form and ribs. When the length of one section has been poured, the pipe is pulled back the same distance. Concrete is allowed 8 hours to settle, then the section is removed, carried through the form and set in place at the other end, ready for the next pour. Crown pour starts from the tunnel end, which is the more recently excavated part, and proceeds towards the initial shaft.

The two-step pouring procedure is used for tunnels with an outside diameter more than 7 ft.

Monolithic pour is used for smaller tunnels and pouring also starts from the end of the tunnel.

5.6.2 Design of the secondary lining:

The permanent lining thickness is determined by taking the least of the three following conditions:

- 1. It must withstand the earth pressure acting on it during its life-time. If it is proven that the lagging does not decay with time, then the state of equilibrium which is reached before concrete pouring will not be modified with time. The pressure on the concrete lining will then be only due to water pressure and its own weight.

However, the earth pressure may act locally when the time allowed for the deformations to take place is not sufficient or when removal of the poling boards or jacking-back of the ribs is performed.

- 2. Thickness must be compatible with technical limitations, such as the necessary clearance between injection pipe and ribs.

- 3. Because of the irregularities in rib settling, a minimum thickness is required to even out the inner surface of the tunnel.

Comments:

According to the conclusions of Deere and Peck (1969), in the case of Edmonton tunnels, all deformations have taken place when the secondary lining is poured. This is an oversimplification, since sections close to the end of the tunnel have not had enough time to fully deform. However, equilibrium is reached for most of the tunnel length and the thickness should be selected on the basis of conditions (2) and (3). Local problems of bad ground may be coped with in the usual way of varying the stiffness of the primary lining (rib spacing, rib type). Procedures involving breaking of the boards or even their removal in order to eliminate protruding ribs, should be avoided.

5.6.3 Technical limitations to concrete thickness decrease

The steel forms used to pour the concrete are rigid and cumbersome; they cannot be adjusted to the irregularities due to differential settlements of the ribs. Those which are protruding excessively into the tunnel, and that would either prevent the passage of the pipe or reduce greatly the local thickness of the concrete lining are jacked back into the ground. This usually breaks up the lagging. Another method is to lower the forms whenever possible or to remove the poling boards without touching the ribs. In all cases, the earth pressure will act locally on the permanent lining. Elimina g these methods implies that a minimum thickness which will allow the form to pass anywhere must be adopted; this results in overdesign where the differential settlements are not important. Figure 5.1 illustrates the problem. Two inches is a minimum clearance between injection pipe and ribs for facility of the operations. This limitation implies a clearance of 6 to 8 inches between the steel form and the ribs. In the case of precast pipes, a practical clearance of 6 inches between ribs and pipe is the best compromise between volume of concrete required and maintenance of an acceptable level of production.

5.6.4 Rate of advance:

Depending on the forms u vert and crown pour or monolithic pour) the rate of advance can reach 100 ft./day but because of unavoidable delays, it averages 80 ft./day. Completion of the permanent lining would take 2 months for a 2,500 ft. tunnel; one month for the invert pour and one month for the crown pour. However, the actual operations may take up to 4 months.

5.7 Conclusions:

In the Edmonton area, the tunnels are temporarily supported by a rib and lagging lining until a permanent concrete lining is installed. With the exception of some sections of the tunnel, where the permanent lining is set up very shortly after completio the excavation, the rib and lagging lining is allowed to deform sufficiently to reach a new state of equilibrium; unless wood decay affects the lagging, the temporary lining supports thereafter the whole of the effective earth pressure, while the concrete lining is only subjected to hydrostatic pressure.

The initial clearance between lagging and ground varies from 0 to 2 inches and the deflections of the ribs may reach 6 inches in diameter, with an unknown part to be attributed to the effects of the jacking forward of the mole.

The role of the concrete lining is to ensure that the tunnel is safe even if the rib and lagging support happens to disappear by decay and to provide a convenient liner for the flow of water inside the tunnel. Its thickness is determined from the point of view of safety with regard to the earth pressure which is calculated according to Terzaghi's arching effect theory. However, one must be aware that a minimum value of the concrete thickness is imposed by construction limitations such as the necessary clearance of 1 inch between the ribs and the operating pipe on the outside of the concrete-pour form.

CHAPTER VI

Edmonton Available Data

6.1 Data available from literature:

6.1.1 Physical properties of the glacial till:

The glacial till in Edmonton, Alberta, has been studied by Christiansen (1970) and Dejong (1971). The grain size distribution is fairly constant and can be considered as an average to be 43% sands, 38% silt sizes and 19% clay sizes. The average liquid and plastic limits are respectively 19% and 12%. The effective parameters vary from 23° to 31.5° for ϕ' and from 4. to 6. psi. for c' according to Christiansen (1970). The approximate total parameters are 24° for ϕ and 2. psi. for c according to Dejong (1971). The natural water content varies from 6% to 13%, with an average value of 10%, and the degree of saturation reaches an average 79% in Edmonton.

For comparison, the results of the tests carried out by Klohn (1965) on the glacial till of Saskatchewan and by Sherif (1973) on the glacial till of Seattle have been considered.

6.1.2 Modulus of elasticity:

The in-situ modulus of elasticity has been found by

Klohn to be approximately 150,000 psi while for the Edmonton till, Dejong found an in-situ value under full overburden pressure of 100,000 psi. However, this value varies with the contact pressure in excess of the overburden pressure, according to Figure 6.1. To interpret this curve, it is necessary that the overburden pressure be known at the depth under study. Letting OP be the overburden pressure and EP the pressure applied in excess of the overburden pressure, the total applied pressure P is equal to the sum of OP and EP:

$$(1)\ P = OP + EP$$

According to Dejong (1971), the soil modulus of elasticity E is a function of the pressure applied in excess of the overburden pressure only (Fig. 6.1):

$$(2)\ E = E(EP)$$

The average depth of a tunnel in Edmonton can be taken as 80 ft., which corresponds to an overburden pressure of 10 ksf. This value will be used in the calculation of the soil reaction coefficient characterising the elastic behaviour of the till.

6.1.3 Soil reaction coefficient calculation:

By definition, the soil reaction coefficient is the force K, expressed in kips, which is necessary to force a 1 sq. ft. rigid plate of 1 inch into the ground. The coefficient may be calculated by using the formula given by Timoshenko and Goodier (1970) for the settlement of a rigid circular plate:

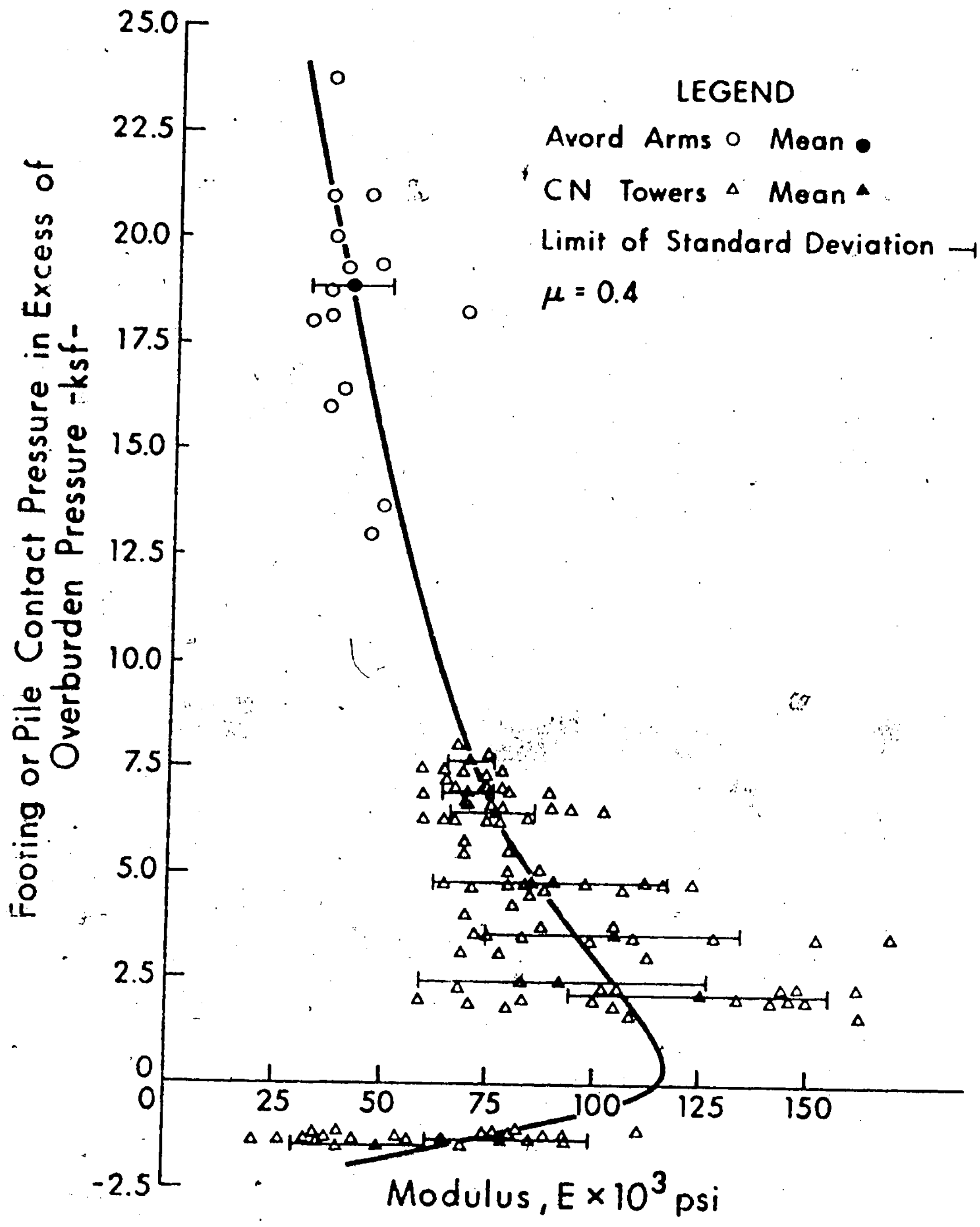


FIG.6.1 - VARIATION OF THE MODULUS OF ELASTICITY OF TILL WITH CONTACT PRESSURE. (AFTER DEJONG - 1971).

(3) $S = \frac{P}{2aE}(1 - \mu^2)$

with S = settlement (in.)

P = total load applied (kips)

a = radius of the plate (in.)

E = soil modulus of elasticity (ksi)

μ = Poisson's ratio.

It is possible to calculate K either by evaluating the modulus of elasticity or by using the S and P results from plate bearing tests.

- Calculation of the soil reaction coefficient from the data of Dejong (1971):

Applying the definition of the soil reaction coefficient to equation (3), with a 1 sq. ft. rigid circular plate pushed 1 inch into the ground and a Poisson's ratio of 0.4 the soil reaction coefficient K may be written as:

$$K \text{ (kips)} = 16.1 \times E \text{ (ksi)}$$

Using an average overburden pressure of 10 ksf, the previous equation is combined with equation (1):

(4) EP (ksf) = 16.1 x E (ksi) - 10. (ksf.)

The system of equations (2) and (4) is solved graphically for E and EP. The curve E(EP) must be extrapolated in order to cut the line represented by equation (4). The limits of E(EP) are its tangent at the last point given by Dejong (1971), i.e. E = 28 ksi, and the vertical line passing through this point. Figure 6.2 represents the extreme shapes that E(EP) can take, along with an average curve. The system of equations (2) and (4) now yields:

$$EP_{maxi} = 441. \text{ kips and } E = 28 \text{ ksi}$$
$$EP_{average} = 57. \text{ kips and } E = 4.2 \text{ksi}$$
$$EP_{mini} = 35. \text{ kips and } E = 2.8 \text{ksi}$$

which in turn gives, according to equation (1), a soil reaction coefficient ranging from 45. kips (absolute minimum) to 451. kips (absolute maximum).

- Calculation of the soil reaction coefficient from the data of Klohn (1965):

A load-settlement curve from Klohn (1965) is given in Fig. 6.3. The loads actually tested in-situ induced settlements which do not exceed 0.3 inch, while the soil reaction coefficient is defined for a 1 inch settlement. However, it is not possible to calculate the soil reaction coefficient directly from the measured data because a greater contact pressure is required for a 1 inch settlement and because the soil modulus of elasticity, and consequently the soil reaction coefficient, vary with the contact pressure as shown by Dejong (1971). It is then necessary to extrapolate the curve Q(S), load versus settlement.

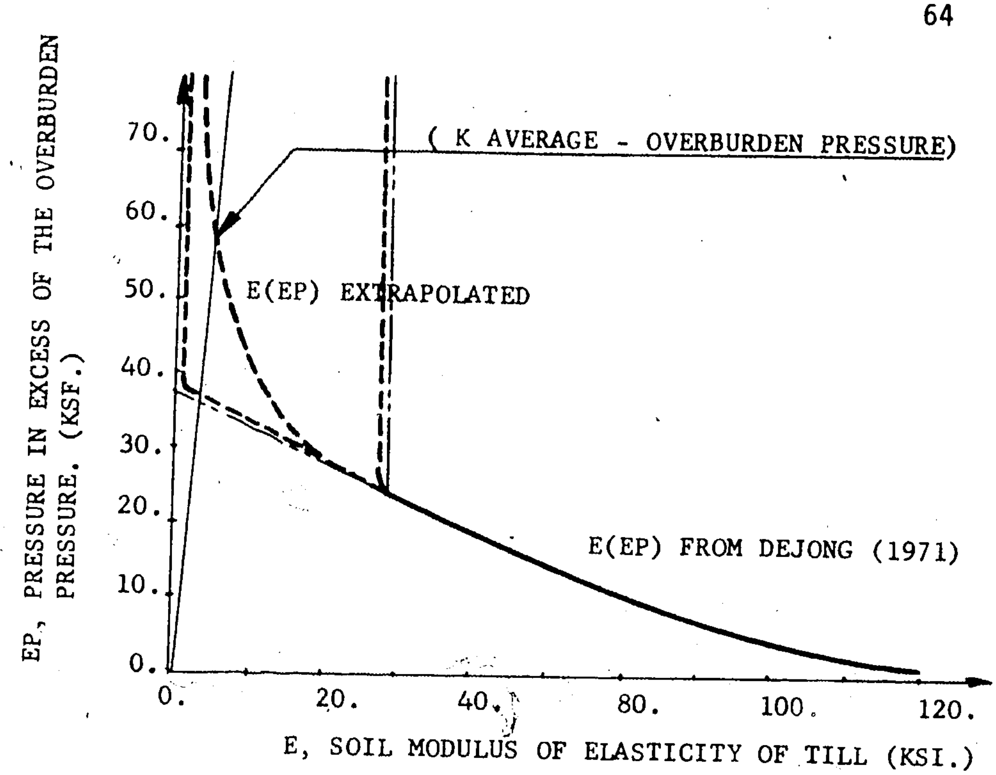


FIG.6.2 - CALCULATION OF THE SOIL REACTION COEFFICIENT FROM THE SOIL MODULUS OF ELASTICITY VALUE.

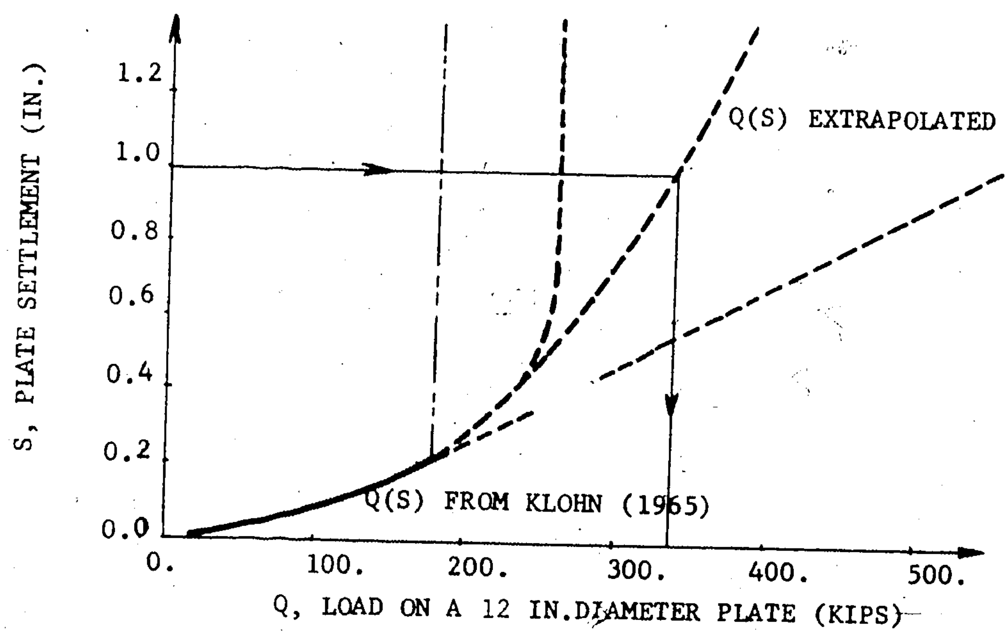


FIG.6.3 - CALCULATION OF THE SOIL REACTION COEFFICIENT FROM PLATE BEARING TESTS.

The extrapolated curve is in a domain limited by the tangent at the measured curve at the last point given, and by an arbitrary curve which consists of part of the circle tangent to the measured curve at this same point and the vertical tangent to this circle as an asymptote. The radius of the circle has been chosen as the smallest probable value of the radius of curvature. This procedure leads to an absolute maximum value of the load required to induce a 1 inch settlement and only to an estimation of the minimum value of the same load.

The vertical line passing through the last given point is the absolute minimum, but it is very improbable that the extrapolated curve has such a shape. An average curve is shown in between the two limiting curves. For a settlement of 1 inch, and using the extrapolated curves, a maximum load of 550. kips, an average load of 334 kips and a minimum load of 250. kips for a 12 inch diameter plate are found. Equation (3) indicates that the ratio of the plate radius to the load inducing a 1 inch settlement in the same material is a constant:

$$(5)\ \frac{p_1}{p_2} = \frac{a_1}{a_2}$$

p_1, total load on plate of radius a_1, for a 1 inch settlement.

p_2, total load on plate of radius a_2, for a 1 inch settlement.

Using this relationship, a soil reaction coefficient varying from a minimum of 282. kips to an absolute maximum of 620. kips, with a probable value of 556. kips, was obtained.

- Calculation of the soil reaction coefficient from the data of Sheriff (1973):

The average settlement due to a uniformly distributed pressure on a flexible circular plate is given by:

$$(6) \quad \Delta L = 0.614 \frac{pxB}{E} \text{ with } \nu=0.5$$

with ΔL average settlement.

p uniform stress.

B diameter of the plate.

E soil dulus of elasticity.

For the same material but different loadings and plate radii, Equation (6) yields:

$$(7) \quad \frac{p_1B_1}{p_2B_2} = \frac{\Delta L_1}{\Delta L_2}$$

Applying this formula to the uniform stresses K_L and K_B inducing settlements of 1 inch on plates with diameters L, corresponding to a 1 sq. ft., plate area, and B:

$$(8) \quad \frac{K_B}{K_L} = \frac{L}{B}$$

By definition, the coefficient K_B is:

$$(9) \quad K_B = \frac{p}{\Delta L}$$

Combining Equations (6), (8), (9), the soil reaction coefficient can be expressed as a function of E and L:

$$(10) \quad K_L = \frac{1}{0.614} \times \frac{E}{L}$$

Sherif and Strazer (1973) then propose a L value which corresponds to a square plate with a surface of 1 sq. ft. This is not correct since the settlement has been calculated for a circular plate. The proper value of L is then 13.54 inches, and leads to the following formula:

$$(11) \quad K_L (\text{ksi.}) = \frac{E}{8.3} (\text{ksi.})$$

With the modulus of elasticity of 33.3 ksi found by Sherif, the soil reaction coefficient is equal to 577. kips.

It can be argued that the plate is in fact behaving as a rigid solid when compared to the soil. With this assumption, the total load acting on Sherif's 2 sq. ft. plate can be backcalculated and, using Equation 5, the load inducing a 1 inch settlement and acting on a 1 sq. ft. rigid circular plate is 596. kips.

The average value of the soil reaction coefficient is calculated from the results of the two previous assumptions concerning the flexibility of the lining. The average soil reaction coefficient is then 586. kips, but, due to the lack of information about the error in the estimation of the soil modulus of elasticity, the precision of this value is unknown.

- Probable value of the soil reaction coefficient in till:

In the previous study, the soil reaction coefficient of the glacial till in Edmonton has been found to be between 282. kips and 620 kips. For comparison, the soil reaction coefficient of a Saskatchewan till is between 45. kips and 451. kips, with an unknown precision concerning the measured data. Although the till properties may vary locally, the comparison is useful to assume a probable value of the soil reaction coefficient in Edmonton, and it is suggested that the soil reaction coefficient of the glacial till in Edmonton be assumed to be between 200. kips and 400. kips.

6.2 Field measurements:

A survey team was set up some years ago to help predict the behaviour of the rib and lagging temporary support and to detect any movement indicative of a potential failure. The measurements consist exclusively of diameter changes of the ribs, surveyed at time intervals dictated by the local behaviour of the lining. Although several test hole plans giving the geologic profile of the subsoil are available from the City of Edmonton Water and Sanitation Department, only two rib profiles were obtained, one illustrating the case of a tunnel buried into the bedrock with an overburden consisting of clay, till and water bearing sand (Fig. 6.4), the other one illustrating the case of a tunnel drilled alternatively into bedrock and till, with an overburden consisting mainly of clay and till (Fig. 6.5). It can be noted that when the water bearing sand layer (Fig. 6.4) increases in thickness to the expense of the till layer, the radial deformations increase consistently. This is consistent with the fact that the arching effect in the till layer cannot develop as fully anymore and the loading on the lining consequently increases. Similarly, a broad correlatior [illegible] observed (Fig. 6.5) when the tunnel passes through till [illegible] l, where the deformations are small and when it passes through bedrock, where the deformations are more important. This can be attributed to the high swelling capacity of the Edmonton bedrock. However, the scattering of the measured data do not allow to predict precisely the deformations under a loading due to a certain geologic profile. The problem is complicated by the fact that the influence of the construction procedures which induce part of the observed deformations is unknown. The quality of the crew, the type of machine used, and the construction practice are the factors determining the displacements attributed to the construction procedures. Rather than trying to predict theoretically these displacements, it seems

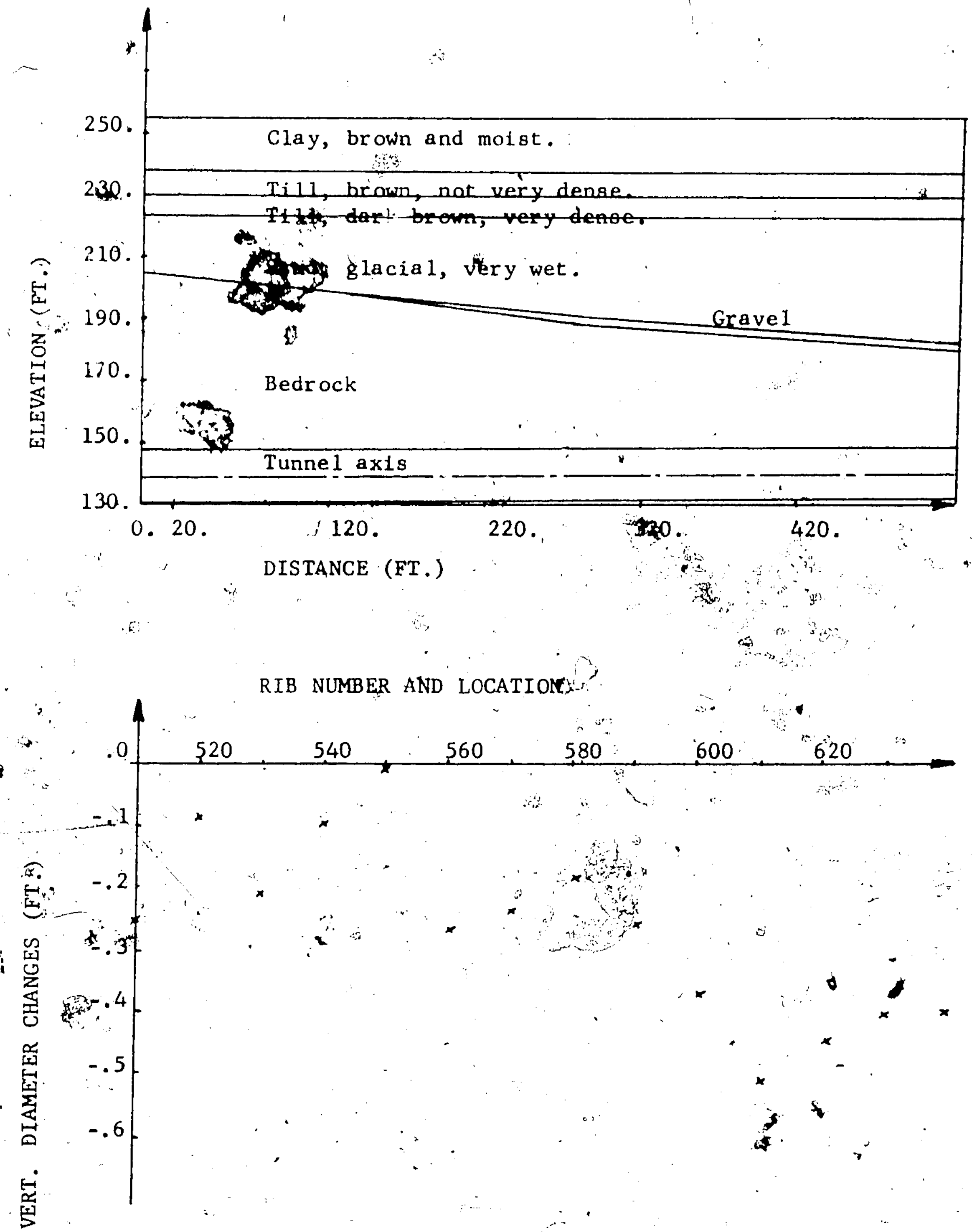


FIG.6.4 - DIAMETER CHANGES AND GEOLOGIC PROFILE OF THE STORM TUNNEL 30AV. AND HIGHWAY 2 TO WHITEMUD CREEK.

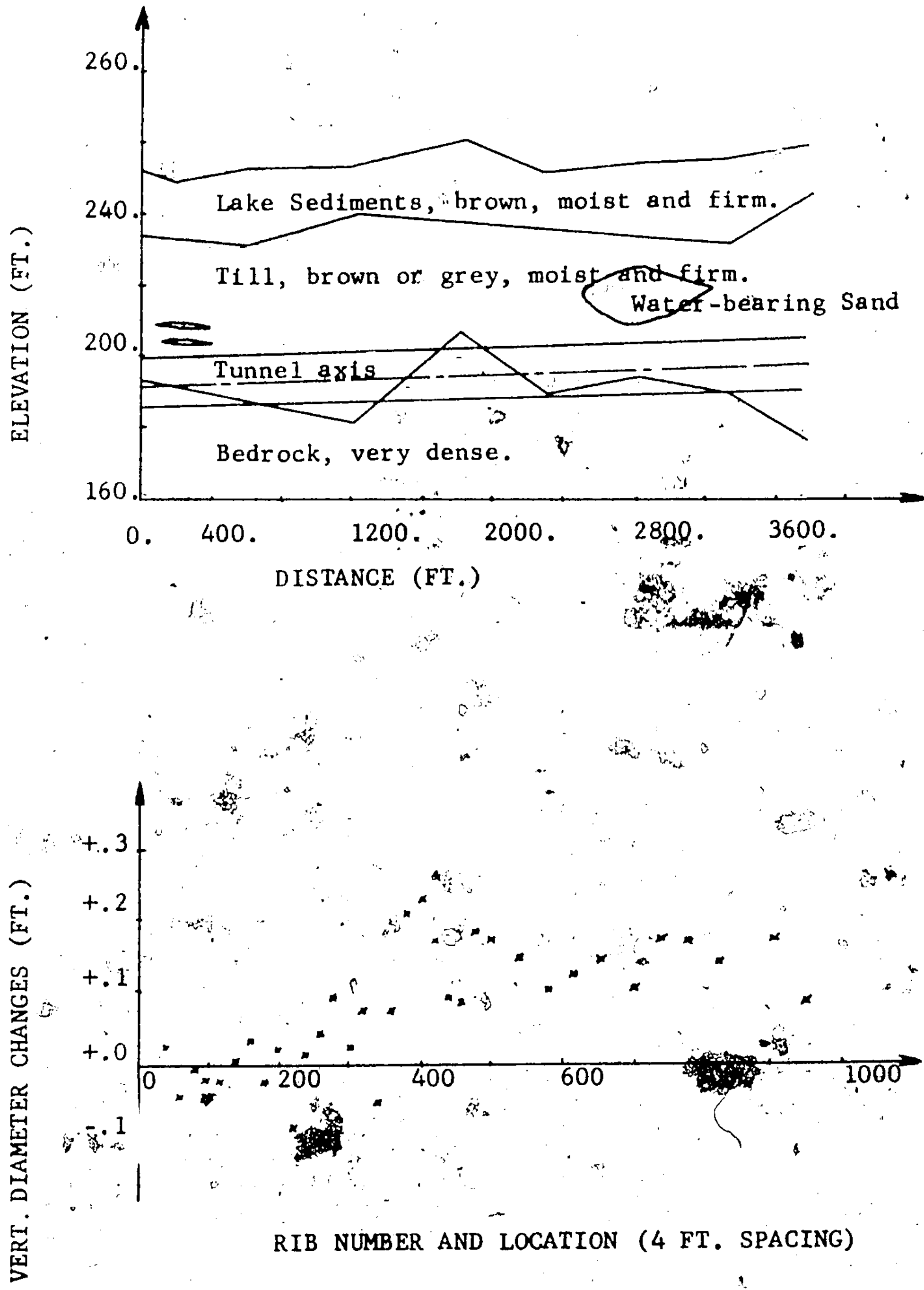


FIG.6.5 - DIAMETER CHANGES AND GEOLOGIC PROFILE OF THE TUNNEL 163St. - 93AV. TO 100AV.

more practical to rely upon data measured in previous tunnels excavated with similar techniques and a similar crew.

The displacements measured in the field are the sum of the displacements due to the construction procedures and the displacements due to the earth pressure. The later displacements can be evaluated from in-situ earth pressure measurements and use of the method of Anders Bull. Hence the displacements due to the construction procedures can be determined for the most common materials in Edmonton, from several case histories indicating the final deformations of the ribs, the geologic profile and the measured earth pressure distribution. Tabulated displacements due to construction procedures and displacements calculated by the method of Anders Bull would lead to the prediction of the final rib displacements. However, at the present time, only the two cases presented in Fig. 6.4 and Fig. 6.5 are available, and their loading distribution is unknown. This emphasizes the need to collect more meaningful and extensive measurements from the Edmonton tunnels.

CHAPTER VII

Relative Importance of Some Factors in the Design of a Tunnel Lining

7.1 Purpose of the study:

The design of a tunnel involves the knowledge of many soil parameters which are known with various degrees of precision. The purpose of the study is to determine the most important of them, thereby helping in the planning of a comprehensive field measurement program and reducing the cost of exploration and construction works.

A tunnel section of the Edmonton area has been arbitrarily chosen as a reference section (Figure 7.1). Everything remaining the same, some parameters were varied and their effects were compared. The tunnel section is lined with a preliminary rib and lagging lining and a permanent concrete lining.

In order to find an upper limit for the moments and thrusts in the lining, three assumptions were made.

- 1. The rib and lagging lining provides support as long as the concrete lining is not poured in place; the deformations of the tunnel were studied with the rib and lagging lining only in a first phase,

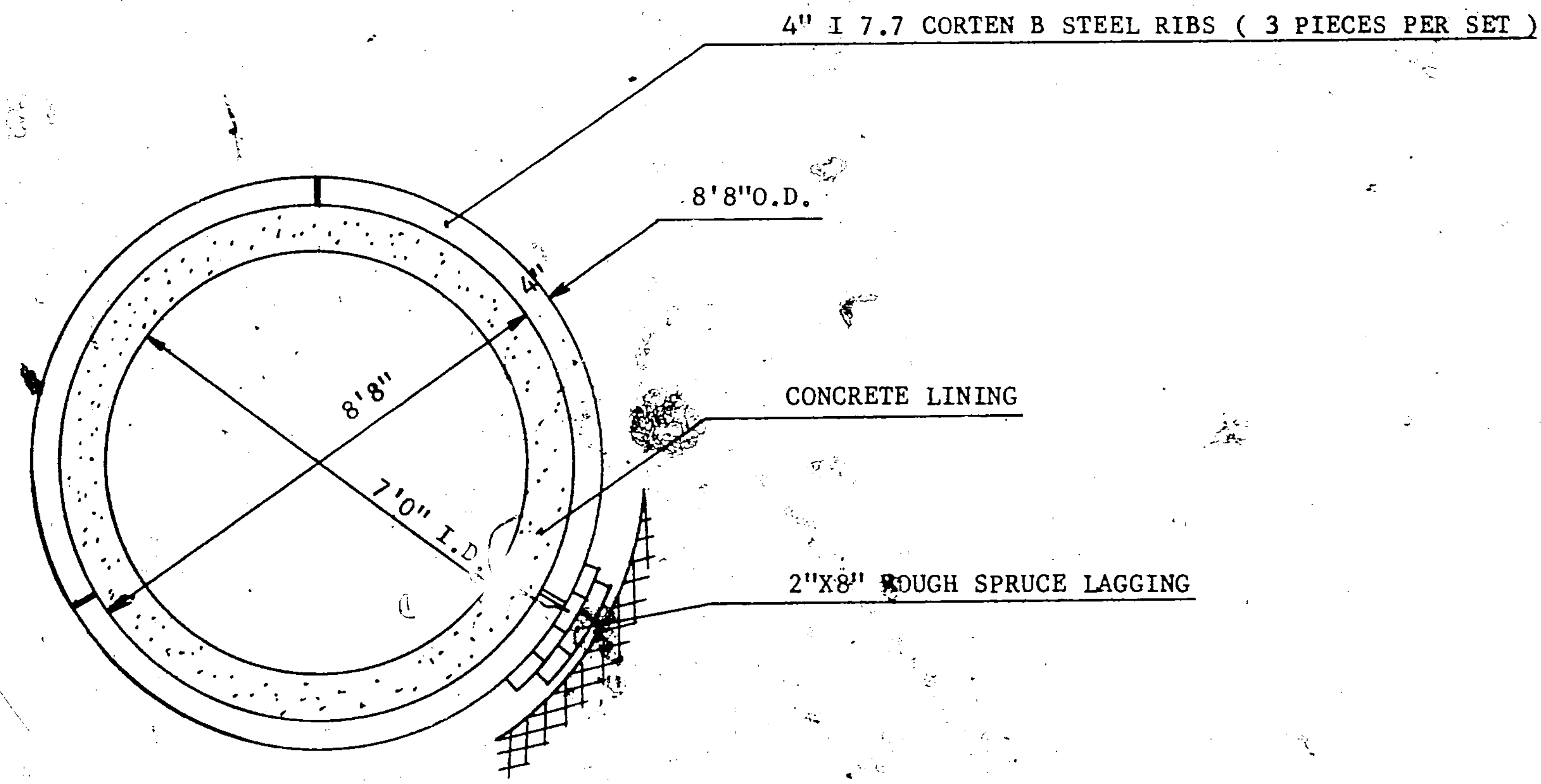


FIG. 7.I - SECTION OF TUNNEL STUDIED IN DETAILS IN CHAPTER 7.

- 2. In the long term, the lagging is assumed to be unreliable because of possible wood decay with time and the concrete lining is considered in a second phase as the only support of the section, with neglect of the effect of the steel ribs within its mass.

- 3. The loading is calculated as if the full overburden pressure were acting on the tunnel, which is the case for shallow tunnels only. Furthermore, the basic principles of Anders Bull's method are applied in the evaluation of the loading. The ground reaction effects are approximated by springs with a stiffness AK. [illegible] surrounding soil is sufficiently disturbed by the boring [illegible] not to transmit any shear forces to the lining. Consequently, [illegible] only tangential loads acting on the lining are due to the dead weight of the lining itself.

It should be kept in mind that the actual loading may vary considerably from these assumptions, with an arching effect usually leading to an earth pressure in the range 15% to 40% of the overburden pressure. The rib and lagging lining seems to provide adequate and reliable support for as long as 30 years, with no decay noticeably affecting the strength of the boards, provided the ground water table does not vary. In the very long term, however, the lagging rate of decay is unknown. At least during 30 years, the loading is divided between rib and lagging and concrete lining, further departing from the assumptions. The results should then be considered as the actual upper limit that the stresses can reach with this tunnel section.

7.2 Rib and lagging lining:

The section which is studied is an 8 ft. 8 in. O.D. tunnel in till supported by a rib and lagging lining using I/7.7 beams of Corten B Steel with an allowable compressive stress of 30,000 psi. The geometric characteristics of the section and the soil and the rib properties are summarized in Table 7.1.

The selection of a rib size is based upon the knowledge of the maximum rib stress which is expected, since the stresses are assumed to be linearly distributed through the section. Hence, because of its meaning in design, the maximum stress in a rib has been chosen as the comparison criterion.

The lagging is assumed to provide a perfect contact with the adjacent ground so that each rib supports the earth pressure on a length of tunnel equal to the spacing of the ribs. A full overburden pressure is applied according to the assumptions in Section 7.1 and a K_o value of 0.67 is assumed.

Rib spacing for the reference section was chosen as 1 ft. and 4 ft. The last value indicates a loading approximately 4 times the one corresponding to a 1 ft. spacing. Accordingly two sets of

TABLE 7.I - REFERENCE SECTION DATA.
RIB AND LAGGING LINING

SYMBOL		
GMA	UNIT WEIGHT OF SOIL	I25. PCF.
GMS	SUBMERGED UNIT WEIGHT	I50. PCF.
GMW	UNIT WEIGHT OF WATER	63.4 PCF.
SK	COEFF. OF EARTH PRESSURE AT REST	.667
RO	TUNNEL OUTSIDE DIAMETER	8.67 FT.
R	RIB MEAN DIAMETER	100. IN.
	SIZE OF A RIB	4"I7.7
AI	MOMENT OF INERTIA, X-X AXIS	6. $IN.^4$
AS	SECTION MODULUS, X-X AXIS	3. $IN.^3$
A	TOTAL SECTION AREA OF RIB	2.2 $IN.^2$
PW.	WEIGHT OF RIB PER FOOT	7.7 LBS/FT.
E	YOUNG'S MODULUS OF ELASTICITY OF RIB	30 000 000 PSI.
	PARAMETERS :	
DEPTH	DISTANCE FROM CROWN TO EARTH SURFACE	30. FT.
SPACNG	RIB SPACING	I.or 4.FT.
HW	DISTANCE FROM GROUND WATER TABLE TO SURFACE	40. FT.
AK	SOIL REACTION COEFFICIENT	43.

LOADING IS COMPUTED FOR FULL OVERBURDEN PRESSURE.

GROUND WATER TABLE VARIATIONS :								
DEPTH OF GROUND WATER TABLE (FT.)	40.	30.	20.	I0.	0.			
CROWN OUTER FLANGE STRESS (KSI.)	-I0.6	-I0.2	-I0.3	-I0.4	-I0.4			
INNER FLANGE STRESS AT JOINT F (KSI.)	-I0.5	-I0.2	-I0.5	-I0.7	-I0.7			
SOIL REACTION COEFFICIENT VARIATIONS :								
SOIL REACTION COEFFICIENT	.I	.7	20.	43.	I20.	203.	325.	45I.
MAXIMUM STRESS (KSI.)	-I2.7	-I2.6	-II.5	-I0.6	- 9.6	-9.2	-8.8	-8.5
RIB SPACING VARIATIONS :								
SPACING (FT.)	4.	3.	2.	I.				
MAXIMUM STRESS (KSI.)	-37.5	-28.7	-I9.8	-I0.6				
ARCHING EFFECT VARIATIONS :								
RATIO EARTH PRESSURE / OVERBURDEN PRESSURE	I.00	.83	.67	.50	.33			
MAXIMUM STRESS (KSI.)	-I0.6	-8.7	-6.8	-4.9	-2.9			

TABLE 7.2 - INFLUENCE OF THE VARIATIONS OF SOME FACTORS ON A RIB AND LAGGING LINING FOR THE REFERENCE SECTION WITH A SPACING OF I FT.

GROUND WATER TABLE VARIATIONS :					
DEPTH OF GROUND WATER TABLE (FT.)	40.	30.	20.	IO.	0.
MAXIMUM STRESS (KSI.)	-37.5	-36.5	-37.6	-38.7	-39.7
SOIL REACTION COEFFICIENT VARIATIONS :					
SOIL REACTION COEFFICIENT	.I	43.	45I.		
MAXIMUM STRESS (KSI.)	-50.3	-37.5	-3I.7		
RIB SPACING VARIATIONS :					
SPACING (FT.)	4.	3.	2.	I.	
MAXIMUM STRESS (KSI.)	-37.5	-28.7	-I9.8	-I0.6	
ARCHING EFFECT VARIATIONS :					
RATIO EARTH PRESSURE / OVERBURDEN PRESSURE	I.00	.83	.67	.50	.33
MAXIMUM STRESS (KSI.)	-37.5	-3I.0	-24.4	-I7.8	-II.3

TABLE 7.3 - INFLUENCE OF THE VARIATIONS OF SOME FACTORS ON A RIB AND LAGGING LINING FOR THE REFERENCE SECTION WITH A RIB SPACING OF 4 FT.

values have been obtained and are summarized in Tables 7.2 and 7.3, for comparison of variations of ground water table, soil reaction coefficient, rib spacing and arching effect.

The maximum compressive stresses of -10.6 ksi and -37.5 ksi obtained for the reference section for a rib spacing respectively of 1 ft. and 4 ft. are used as reference stresses to evaluate in percentage the variation of stresses.

The ground water table varied from the ground surface down to a depth below the tunnel invert at 38.66 ft. As expected, most of the maximum stresses occurred at the crown or joint J because the deformations of the lining at this joint are usually the most important. However, in the case of a 1 ft. rib spacing, the maximum rib stresses occurred slightly over the springline at joint F. The results are plotted in Fig. 7.2. It may be noted that the effect of the ground water variations increases with the loading. The actual shape of the curves has little importance because of the very small range of variations of the stresses which does not exceed 10% of the maximum reference stress for both spacing while the groundwater table varies from the tunnel invert up to the surface.

The coefficient of soil reaction will vary with the material, the depth of the tunnel and the hydrologic conditions. For the purpose of the parametrical study only, the soil reaction coefficient has been made to vary beyond its probable range of 200 kips-400 kips for a glacial till in Edmonton (see Chapter 6). In this study, it varies from 0.1 kips to 451. kips and the results are plotted in Fig.7.3. With a 1 ft. rib

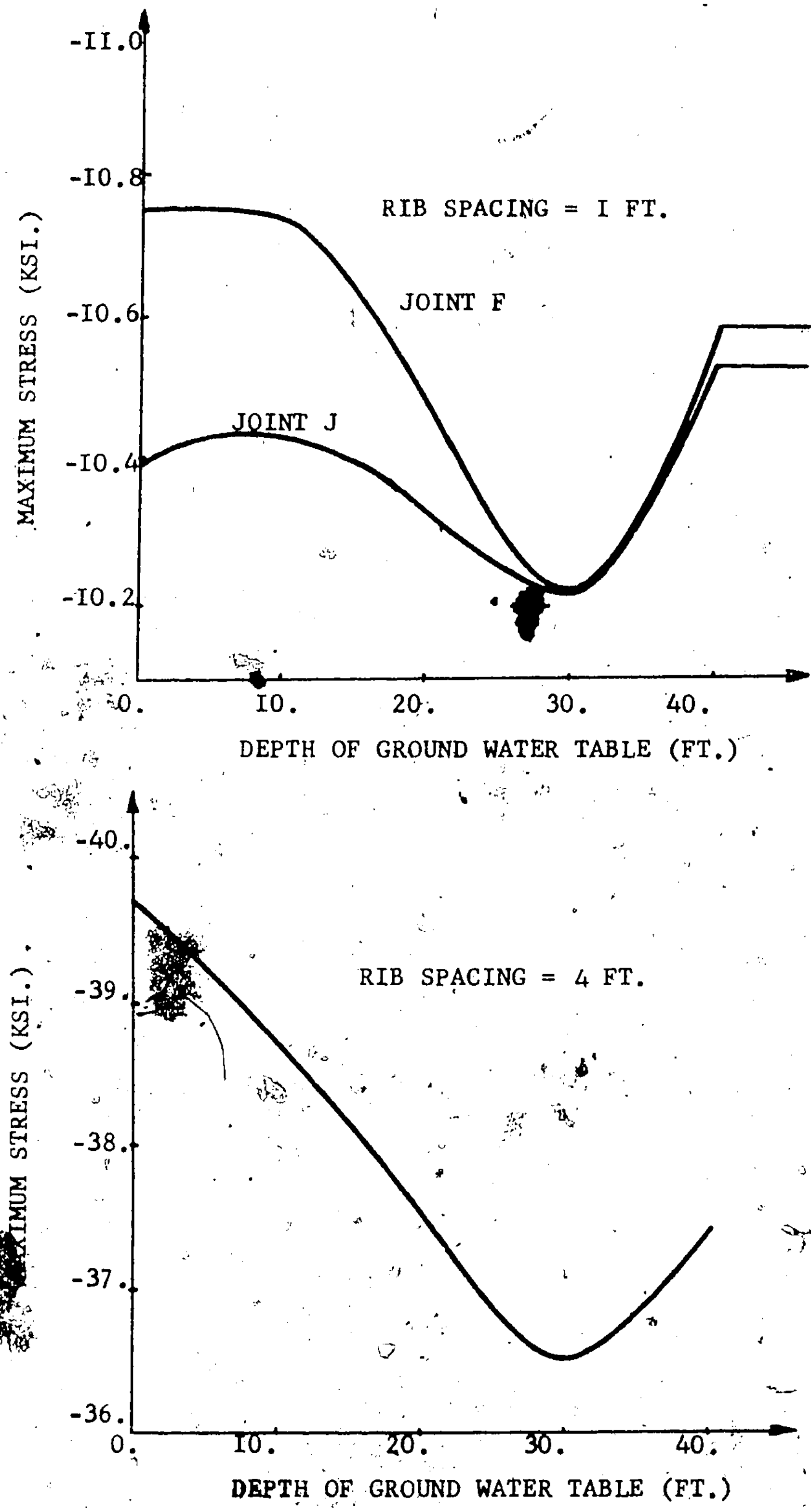


FIG. 7.2 - INFLUENCE OF GROUND WATER TABLE VARIATIONS ON A RIB AND LAGGING LINING FOR THE REFERENCE SECTION.

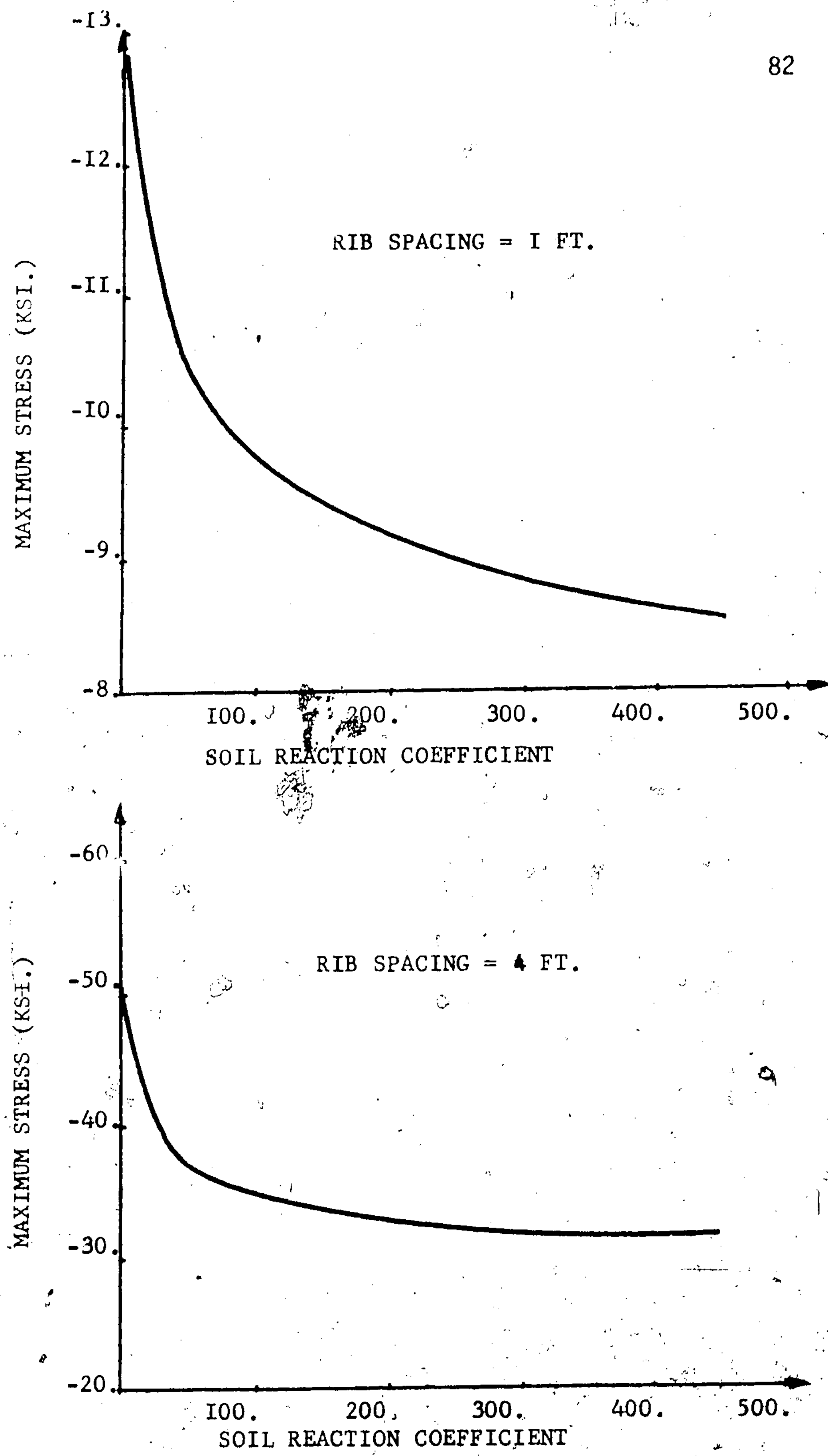


FIG. 7.3 - INFLUENCE OF SOIL REACTION COEFFICIENT VARIATIONS ON A RIB AND LAGGING LINING FOR THE REFERENCE SECTION.

spacing, a very sharp decrease of 22% in stresses may be noticed in the low range from 0.1 to 120.,.while a moderate decrease in stresses of 12% is experienced from 120. to 451. A similar curve, with more pronounced variations, is obtained with a 4ft. rib spacing. The shape of the curves indicates that the value of the coefficient of ground reaction loses of its importance when the soil is sufficiently stiff as in till, since in this case an approximate knowledge of it is satisfactory for determining the stress level. This conclusion has a direct implication on a field measurement program which should be aimed at obtaining a gross value of the soil reaction coefficient for a stiff soil. For a variation of the coefficient of ground reaction from 0.1 to 45.1 kips, the corresponding total stress variation from the maximum reference stresses is in both cases less than 40%.

The rib spacing variations have a very important effect on the rib stresses as illustrated in Fig. 7.4. For all practical purposes, the relationship between the maximum stress and the rib spacing is linear, the effect of the lining weight being negligible. From a 4 ft. spacing to a 1ft spacing, the maximum stress decreased by 72%. This emphasizes the critical importance of rib spacing determination in the design. Although it is possible to evaluate a certain set of values corresponding to the most common soils to be found in a particular area, in-situ changes of the rib spacing should be allowed in order to provide for the unexpected local irregularities of the subsoil.

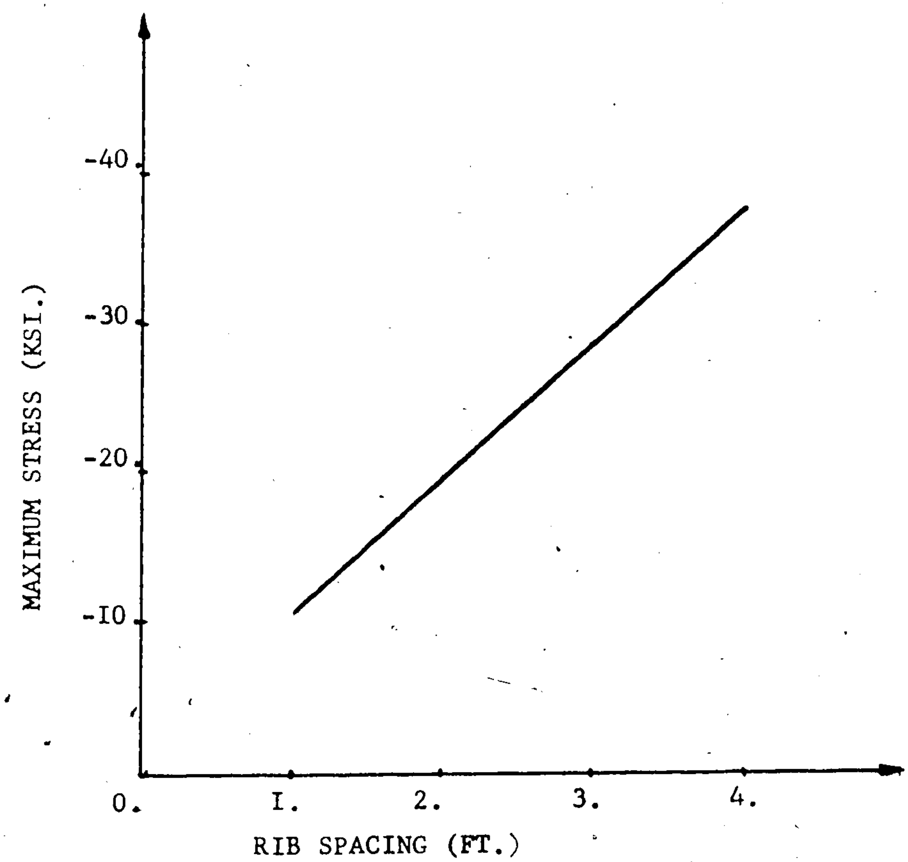


FIG. 7.4 - INFLUENCE OF SPAC[illegible] VARIATIONS ON A RIB AND LAGGING LINING FOR THE REFERENCE SECTION.

The arching effect reduces the pressure on the tunnel to a fraction of the overburden pressure. In order to use an automatic computation of the loading, this reduction of pressure has been achieved by decreasing the depth of the tunnel while calculating the loading for the full overburden pressure under this new depth. The effect is similar to the rib spacing effect, although theoretically of different nature but the effect of the lining weight is once again negligible and the relationship maximum stress - arching effect can be considered linear (Fig. 7.5). When reducing the earth pressure to 1/3 of the overburden pressure the maximum stress decreased by an average 70% for both sets of spacing values.

To summarize and compare conveniently the results, the relative importance of the parameters is shown in Fig. 7.6. Each column represents the relative variation $\Delta \sigma/\sigma$ of the stresses in percentage for a given parameter varying between the values indicated at top and bottom of the column. The ground water table varies from the tunnel invert to the ground surface.

For both rib spacing values, the ground water table variations have very little effect while the variations of the three other parameters influence the stresses considerably. However, it should be noticed that their relative importance varies somewhat with the loading, as illustrated for the spacings 1 ft and 4 ft.

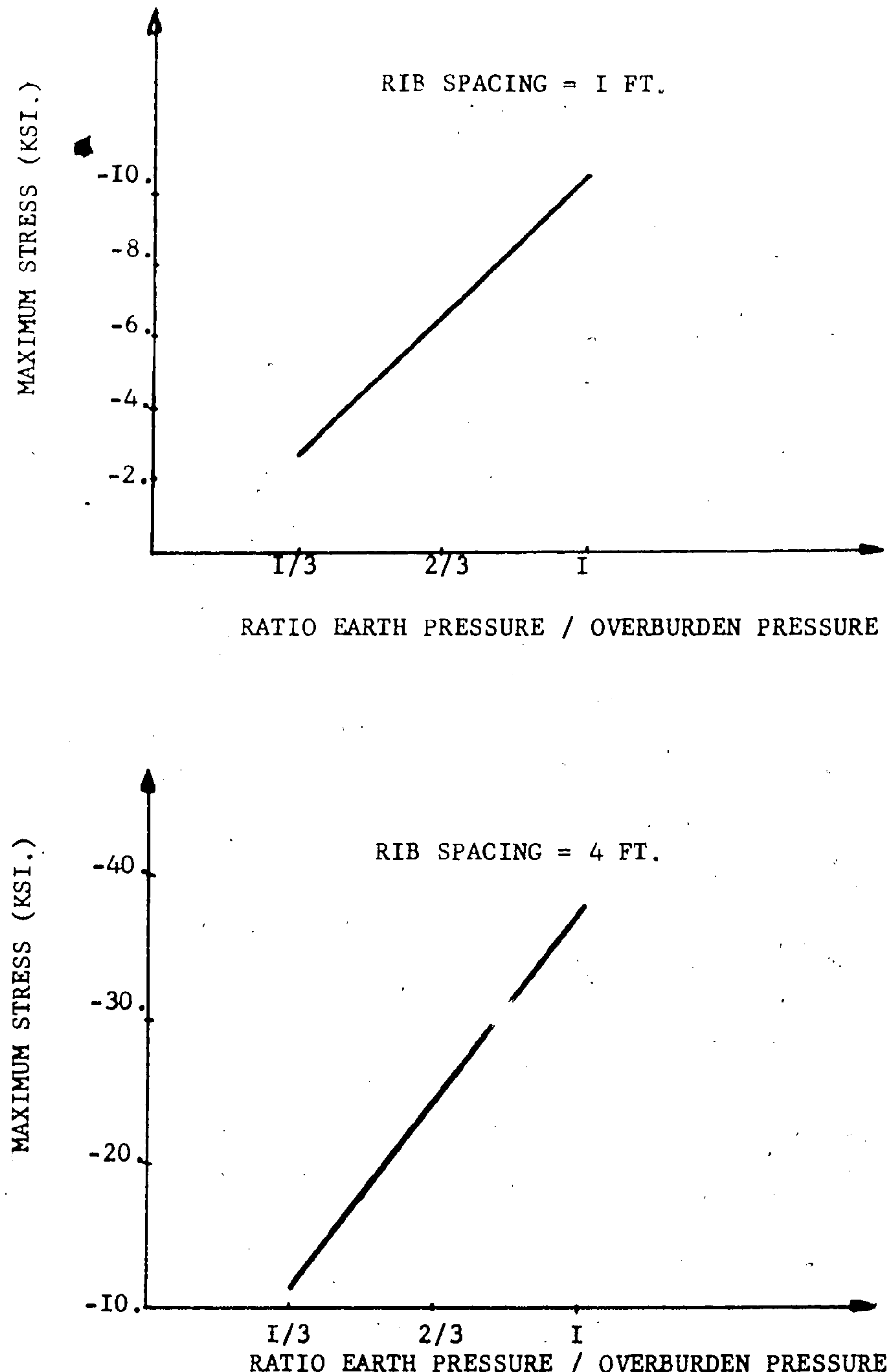


FIG. 7.5 - INFLUENCE OF ARCHING EFFECT VARIATIONS ON A RIB AND LAGGING LINING FOR THE REFERENCE SECTION.

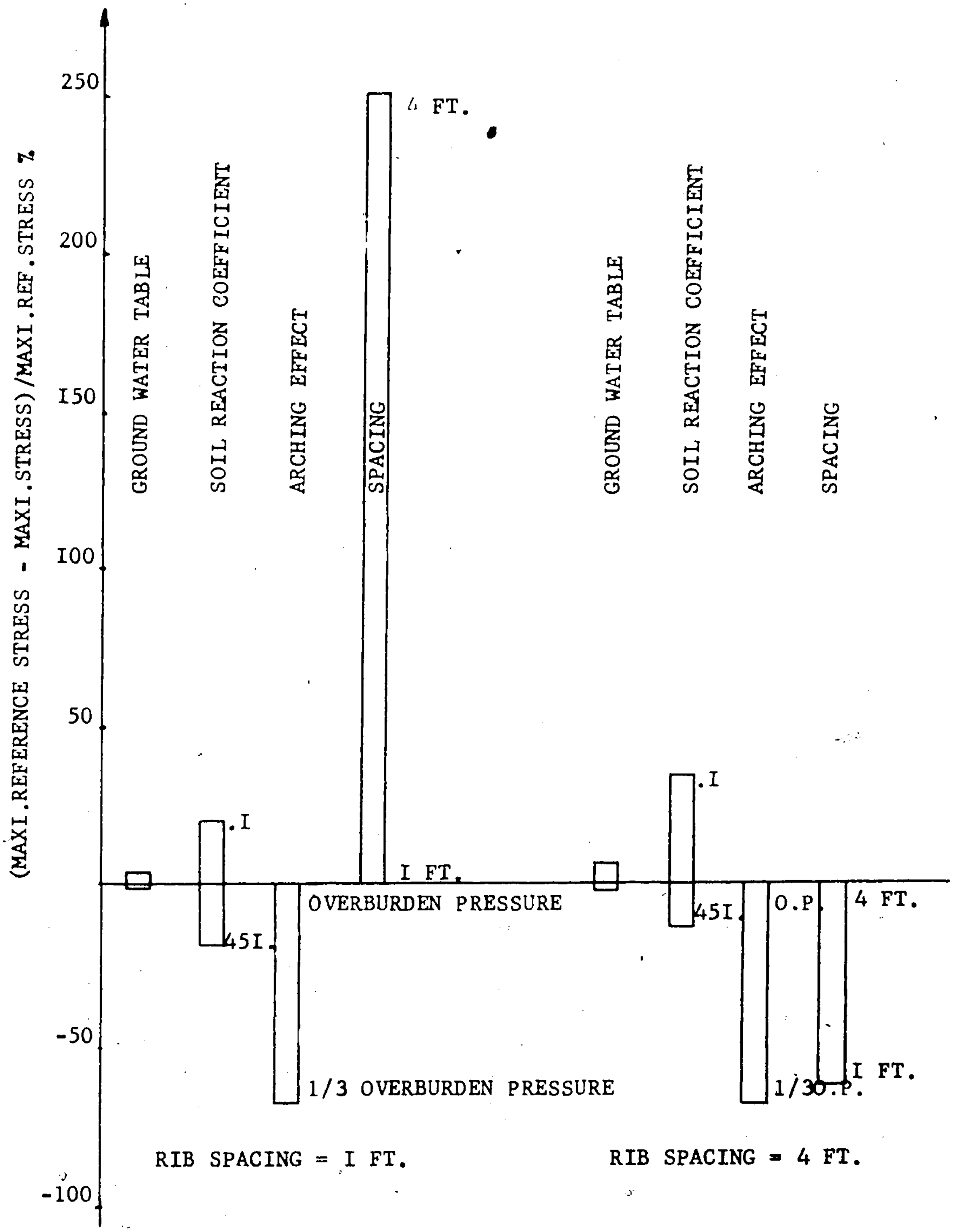


FIG. 7.6 - COMPARISON OF THE INFLUENCE OF SOME FACTORS ON A RIB AND LAGGING LINING FOR THE REFERENCE SECTION.

Clearly soil reaction coefficient, arching effect and rib spacing are the dominant factors determining the rib stresses. If the results are discussed in the case of the Edmonton till, further conclusions can be reached: the soil reaction coefficient for till is likely to be in the range 200. - 400. - Because of the flatness of the curve Maximum stress - Soil reaction coefficient in the upper range, an approximate knowledge of the soil reaction coefficient will be adequate to determine the maximum stress without significant error.

On the other hand, a precise knowledge of the magnitude of arching is required because of its great importance on the stress level. The available data in Edmonton does not allow one to evaluate properly the extent and the value of arching in till. Such knowledge, which is crucial for design as demonstrated in this study, should be obtained from a comprehensive field measurement program.

Assuming all parameters known, the designer has to select a proper rib spacing whose value is determinant on lining behaviour. Revision of this choice should be allowed in the field as the work progresses to meet the local subsoil non-homogeneities.

7.3 Concrete lining:

The section studied is the same 8 ft. 8 in., O.D. tunnel which was used for comparative study with a rib and lagging lining.

The tunnel is now assumed to be supported by a non-reinforced concrete lining. The soil properties are similar to the previous ones and the geometric characteristics refer to a tunnel ring 1 ft. wide. The input data for the reference section - concrete lining - is listed in Table 7.4.

One procedure to design a non-reinforced concrete beam is to apply certain load factors to live loads and dead loads and to design the beam at rupture. The previous design practice was to assume a linear distribution of the stresses through the section and to calculate the beam so that the maximum compressive stress would never be greater than the allowable compressive stress. Although the latter method leads to more conservative design, the maximum stress occurring in a beam can still be considered as a meaningful criterion for design. Since the purpose of this chapter is essentially to compare the magnitude and the trends, as opposed to the actual values, due to the variations of some parameters, the maximum stress will be used as the comparison criterion.

The loading is taken as the full overburden pressure for reasons given in Section 7.1. The K_0 value is assumed to be 0.667 and soil and concrete lining are in ideal contact.

A summary of the results is presented in Table 7.5. The maximum compressive stresses occurred either at the crown outer flange or joint J, or on the inner flange slightly over the springline at joint F. The curves represent the maximum stresses versus the parameters without mention of the joint where the maximum stresses occurred.

TABLE 7.4 - REFERENCE SECTION DATA -
CONCRETE LINING

SYMBOL			
GMA	UNIT WEIGHT OF SOIL	125	PCF.
GMS	SUBMERGED UNIT WEIGHT	150	PCF.
GMW	UNIT WEIGHT OF WATER	63.4	PCF.
SK	COEFF. OF EARTH PRESSURE AT REST	.667	
RO	TUNNEL OUTSIDE DIAMETER	8.67	FT.
T	THICKNESS OF THE LINING	8.	IN.
R	LINING MEAN DIAMETER	96.04	IN.
B	WIDTH ALONG TUNNEL AXIS, FOR AI,AS,A.	12.	IN.
AI	MOMENT OF INERTIA, X-X AXIS	512.	$IN.^4$
AS	SECTION MODULUS, X-X AXIS	128.	$IN.^3$
A	TOTAL SECTION AREA WITH WIDTH B	96.	$IN.^2$
PW	WEIGHT PER FOOT OF PERIMETER WITH WIDTH B	45.14	LBS/FT.
E	YOUNG'S MODULUS OF ELASTICITY FOR CONCRETE	3 000 000	PSI.
	PARAMETERS		
DEPTH	DISTANCE FROM CROWN TO EARTH SURFACE	30.	FT.
THICKNESS	THICKNESS OF CONCRETE LINING	8.	IN.
HW	DISTANCE FROM GROUND WATER TABLE TO SURFACE	40.	FT.
AK	SOIL REACTION COEFFICIENT	43.	

LOADING IS COMPUTED FOR FULL OVERBURDEN PRESSURE.

GROUND WATER TABLE VARIATIONS :									
DEPTH OF GROUND WATER TABLE (FT.)	0.	10.	20.	30.	32.	34.33	38.66	60.	
CROWN OUTER FLANGE STRESS (KSI.)	-.251	-.257	-.263	-.269	-.282	-.291	-.289	-.289	
MAXIMUM STRESS - JOINT J - (KSI.)	-.258	-.263	-.267	-.271	-.283	-.291	-.290	-.290	
SOIL REACTION COEFFICIENT VARIATIONS :									
SOIL REACTION COEFFICIENT	.1	.7	8.1	43.	64.5	120.	203.	325.	451.
MAXIMUM STRESS (KSI.)	-.304	-.304	-.300	-.290	-.287	-.283	-.272	-.254	-.244
JOINT	J	I	J	F	F	F	F	F	J
THICKNESS VARIATIONS :									
THICKNESS (IN.)	10.	8.	6.	4.	2.				
MAXIMUM STRESS (KSI.)	-.211	-.290	-.432	-.698	-1.341				
ARCHING EFFECT VARIATIONS :									
RATIO EARTH PRESSURE / OVERBURDEN PRESSURE	1.00	.83	.67	.50	.33	.17			
CROWN OUTER FLANGE STRESS (KSI.)	-.289	-.232	-.176	-.119	+.063	+.006			
MAXIMUM STRESS (KSI.)	-.290	-.236	-.182	-.127	-.096	-.073			

TABLE 7.5 - INFLUENCE OF THE VARIATIONS OF SOME FACTORS ON A CONCRETE LINING FOR THE REFERENCE SECTION.

The ground water table varied from the surface to below the tunnel invert, 38.66 ft. below the surface. This variation induced a total stress variation of -11% from the maximum reference stress of -0.290 ksi. (Fig. 7.7), while the soil reaction coefficient varying from the arbitrary value 0.1 to the probable maximum value 451. for till induced a stress variation +5% to-16% from the maximum reference stress (Fig. 7.8). As expected, the stresses decrease when the soil becomes stiffer, since deformations and moments are reduced.

When studying the concrete thickness influence, no attention was paid to the technical problems which would be involved when building a concrete lining only 2 inches thick. This study just aims at determining the effect of the thickness on the stress level, and the 2 in. value should be considered only as an assumption for the model. Fig. 7.9 shows the overwhelming importance of the concrete thickness, especially in the low range values. The total thickness variation from 10 in. down to 2 in. would bring about a stress variation of -27% to+ 362% from the maximum reference stress. However, when limiting the practical range of variations to 10 in. - 4 in., the stresses reach -27% and + 75% variations from the maximum reference stress.

Limiting the minimum thickness to 4 in. is necessary because the steel forms used in concrete pouring cannot be inserted between the ribs because of their width. The minimum thickness of

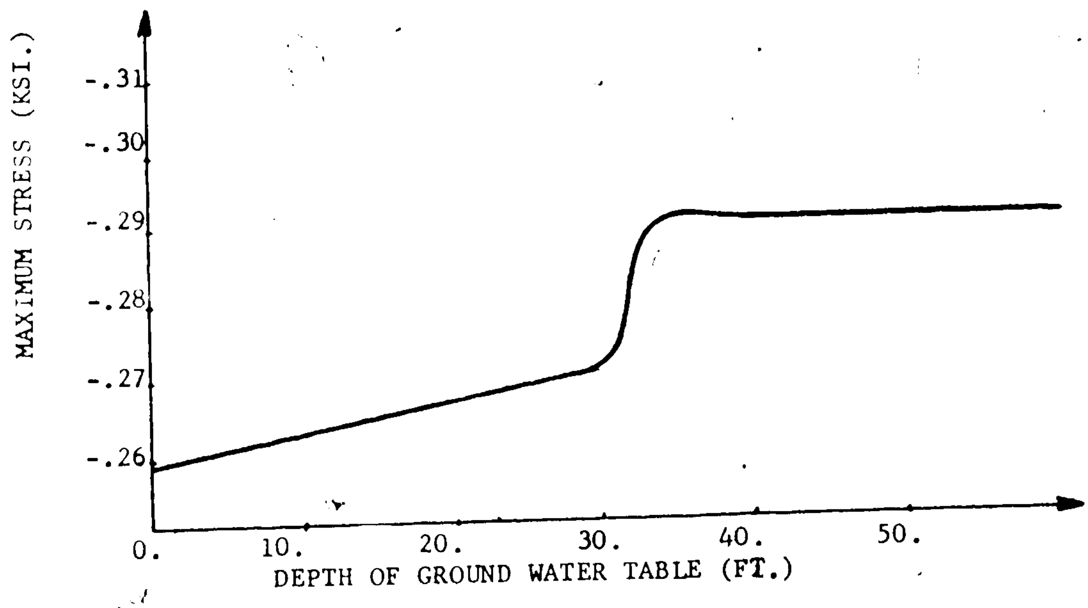


FIG.7.7 - INFLUENCE OF GROUND WATER TABLE VARIATIONS ON A CONCRETE LINING FOR THE REFERENCE SECTION.

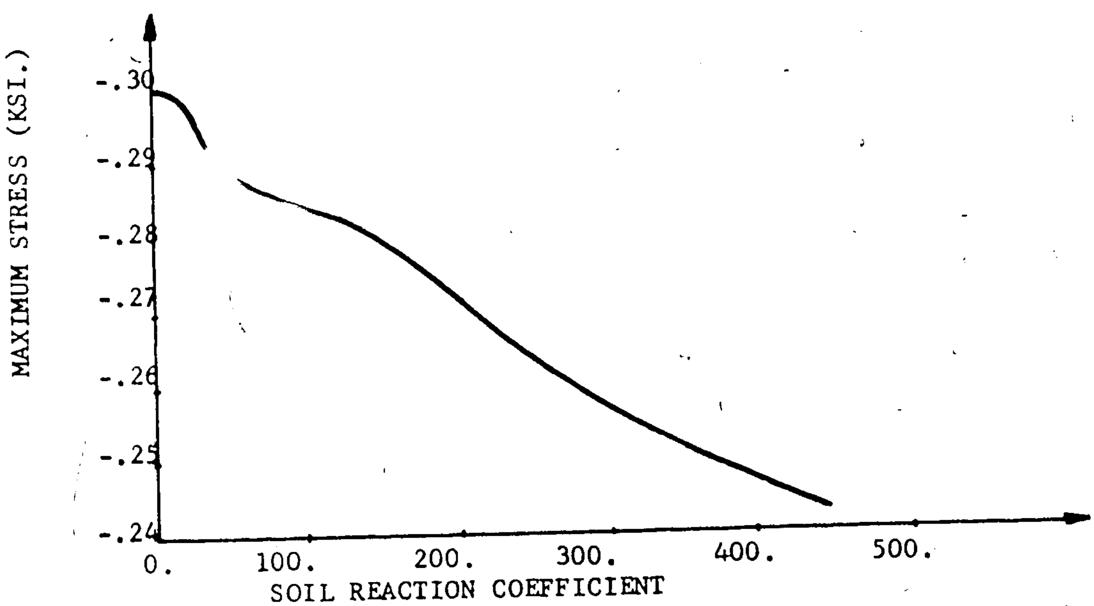


FIG.7.8 - INFLUENCE OF SOIL REACTION COEFFICIENT VARIATIONS ON A CONCRETE LINING FOR THE REFERENCE SECTION.

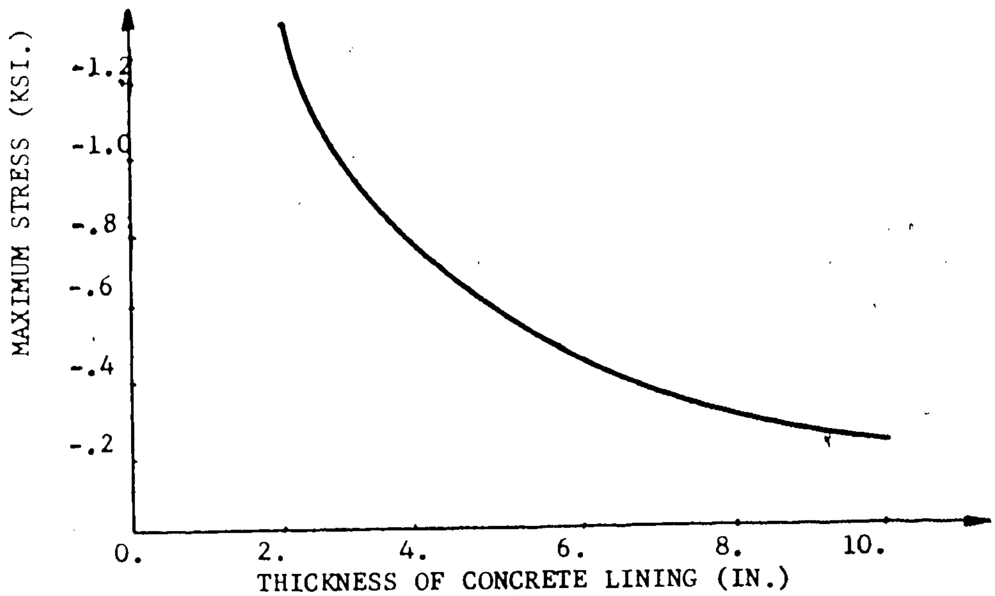


FIG.7.9 - INFLUENCE OF THICKNESS VARIATIONS ON A CONCRETE LINING FOR THE REFERENCE SECTION.

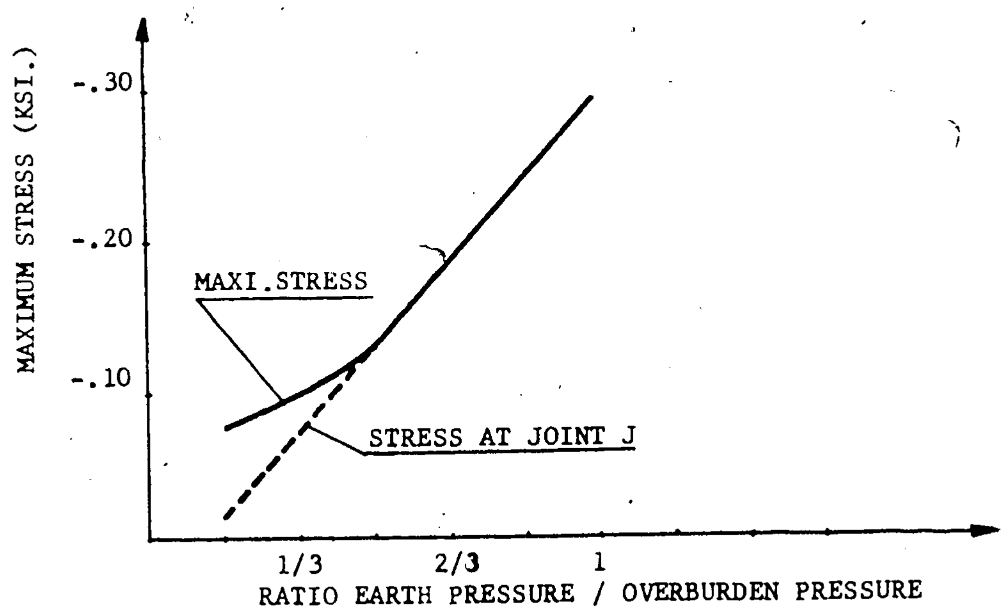


FIG.7.10 - INFLUENCE OF ARCHING EFFECT VARIATIONS ON A CONCRETE LINING FOR THE REFERENCE SECTION.

the lining for one reference section is then equal to the depth of the ribs, 4 in.

As in the rib and lagging case, the effect of arching has been studied by varying the depth of the tunnel and calculating moments and thrusts for the new full overburden pressure. Results are plotted in Fig. 7.10. The relationship depth - maximum stress is linear when the arching effect is assumed to be more than half the overburden pressure. However, from 1/6 to 1/2 this value, the maximum stresses have been found at joint D just below the spring-line, instead of at joint J, and the curve is not linear. This is of importance since the arching effect is probably in the range of 20 - 50% of the overburden pressure. Because of a flatter curve in the low range it may not be required to determine it with as much precision as in the case of a rib and lagging lining.

For convenient comparison of the influence of the previous factors, the variations have been summarized in Fig. 7.11. The arching effect is clearly the factor dominating the design and this stresses again the importance of correctly evaluating it. Choice of the thickness of the lining is also crucial. It should be noticed however that saving 2 in. of concrete on a thick lining does not change considerably the stresses while saving 2 in. on a thin lining might be critical. The curve maximum stress versus concrete thickness should be carefully computed in order to determine whether the proposed reduction in thickness is safe or not.

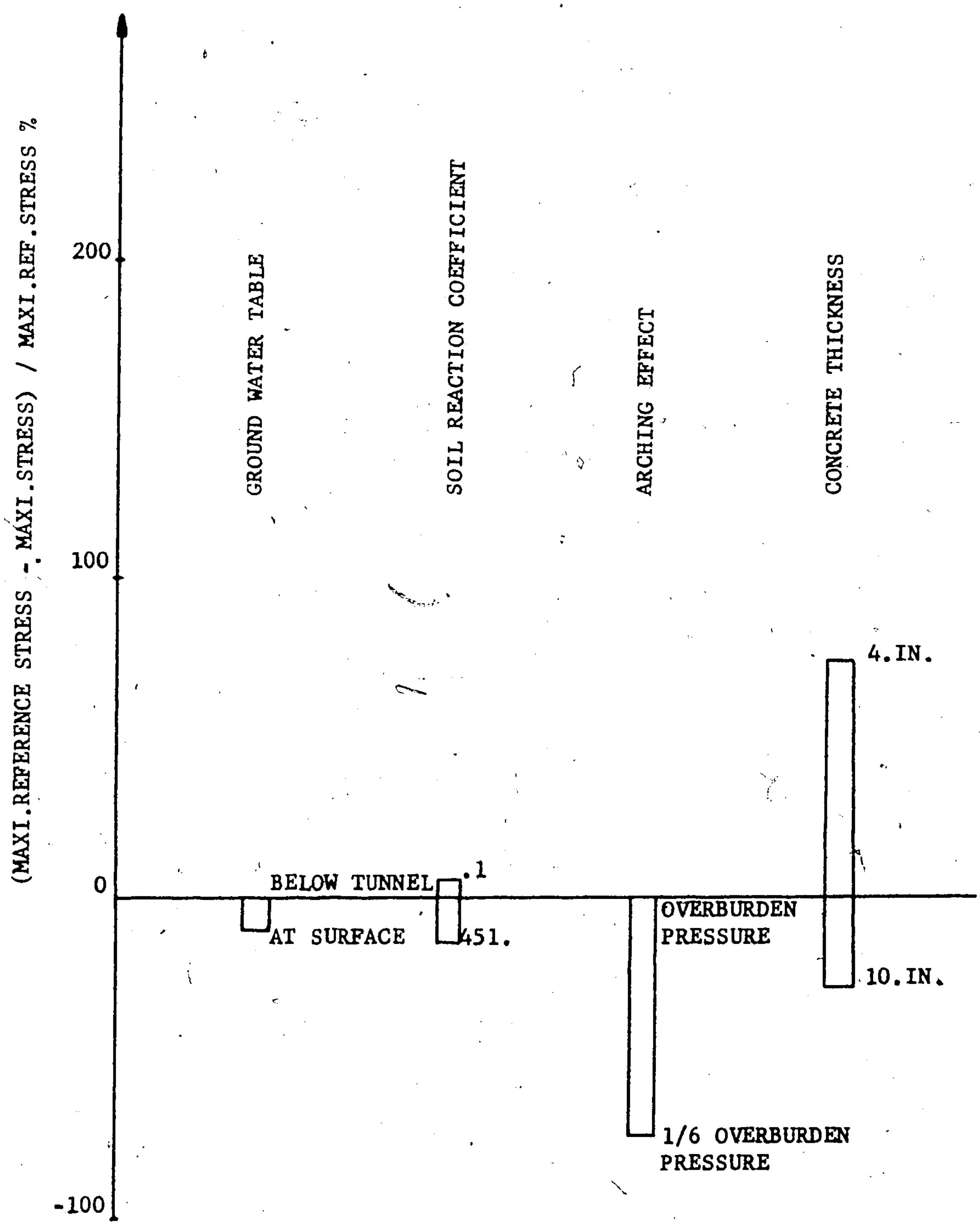


FIG.7.11 - COMPARISON OF THE INFLUENCE OF SOME FACTORS ON A CONCRETE LINING FOR THE REFERENCE SECTION.

7.4 Conclusions:

The previous chapter presents a comparative study of the influence of some factors on the design of a lining. A rib and lagging lining and a non-reinforced concrete lining have been studied separately.

The ground water table effect has been found to be small, with less than 10% of stress variation when the water table varies from the surface to a depth below the tunnel invert.

In the range of values 200. - 400. units to which the soil reaction coefficient for till likely belongs, an approximation of this coefficient with 20% error yields the stress level with 3% error only.

However, in the range of values 15% - 50% of the overburden pressure, which represents the probable domain of variation of the earth pressure, an approximation of the arching effect with the same 20% error yields the correct stress level with 30% error in the case of a rib and lagging lining, and 10% only in the case of a concrete lining.

This illustrates that only a gross knowledge of the soil reaction coefficient is necessary to determine the stresses with reasonable accuracy, while a precise knowledge of the arching effect is required for the same purpose.

In the case of a rib and lagging lining, emphasis in the design should be put on the selection of a proper rib spacing, since a difference of 1 ft. may well mean the difference between safe and unsafe design. This value, however, must be checked constantly in the field to allow for local subsoil non-homogeneities.

For a non-reinforced concrete lining, the selection of the concrete thickness might be critical if a thin lining is projected. The precise knowledge of the variations of the maximum stresses versus the concrete thickness is required for a proper design. Although the final design should be based on a calculation of the ring at rupture, the above curve would indicate the approximate values of the thickness for which safety is critical.

CHAPTER VIII

Example of application : design of a tunnel

8.1 Description of the case and loading assumptions:

The problem is to design a temporary rib and lagging support and a permanent concrete support for a tunnel excavated in till. The case corresponds to the tunnel located at 30A Avenue, from Whitemud Creek to Calgary Trail. A geologic profile and a rib profile are given in Fig. 6.4. The 19.06 ft. O.D. tunnel is located at an average depth of 115. ft. and the specific weight of the overlying soil is taken as 125 psf. The soil reaction coefficient is assumed to be 200. units. The parametrical study (Chapter 7) showed that similar results are expected when the soil reaction coefficient is between 200 and 400 units. Because of the lack of data in the Edmonton area, several assumptions must be made in order to define the loadings on the temporary and permanent supports. The rib and lagging lining is assumed to be subjected to the earth pressure due to the arching effect, calculated according to Terzaghi's theory. Later on, when the concrete lining has been set up, the lagging might decay and the temporary support becomes inefficient. The loading may also revert from the arching state to the full overburden pressure. Consequently, it is assumed that the concrete lining must be designed to withstand the full overburden pressure.

For tunnels at great depths, Terzaghi proposed to calculate the vertical stress by the formula:

$$\sigma_v = \frac{B(\gamma - 2c/B)}{2K\tan\phi}$$

with

$$B = 2\left[\frac{O.D.}{2} + O.D. \times \tan\left(45° - \frac{\phi}{2}\right)\right]$$

O.D. outside tunnel diameter

c, ϕ total parameters of till

γ unit weight of soil

K coefficient of earth pressure at rest.

For simplicity, the arching effect is assumed to develop in the 40 ft. thick till layer only.

Given : Depth = 115 ft.

γ = 125 psf.

O.D. = 19.06 ft.

K_o = 1.

c = 2 psi and ϕ = 24° (Dejong 1971)

the calculation leads to an arching pressure of 4.08 ksf and a total overburden pressure of 14.37 ksf.

8.2 Preliminary design:

With the loading determined as in Section 8.1, the thrust at the springline of the ribs can be calculated provided a rib spacing is chosen. The usual 4 ft. rib spacing is used in this study and the thrust reaches 150.03 kips. With a 6x6WF25 Corten B steel rib, the factor of safety for the thrust is 1.47- The maximum moment can be approximately evaluated by the following formula:

$$\text{Max. moment} = \frac{3EI \times \Delta_a}{a^2} \quad \text{(Deere, Peck et al 1969)}$$

E Young's modulus of lining

I moment of inertia of lining

Δ_a radial deformation

a neutral radius of lining

The deflection due to the earth pressure never exceeds 1% of the diameter (Deere, Peck et al 1969).. With an average assumed deflection of 0.5% the diameter, the maximum moment reaches 198 kips x in. The maximum and minimum stresses are now calculated from the values of thrust and moment.

The maximum compressive stress in the ribs reaches -32.9 ksi, compared to -30.0 ksi allowable compressive stress for Corten B steel. Since the stresses are only grossly approximated by the previous calculations, this result must not be considered as a criterion to reject the use of 6x6WF25 ribs with a rib spacing of 4 ft., which as a preliminary design of the temporary support is satisfactory.

For the concrete lining, the thrust reaches 137. kips per foot of tunnel. In the following calculations, the concrete lining is designed according to the maximum allowable stress concept. Although this approach leads to a conservative design, its simplicity of application with the results of the method of Anders Bull determined its selection. However technical limitations usually prevail and for the preliminary design, the City of Edmonton 12 in. thick concrete liner is considered. With an assumed radial deflection of 0.01% of the diameter, the maximum moment [illegible] calculated by the previous formula and reaches 13.3 kips x in. [illegible] the design approach which has been chosen, the stress distri[illegible] assumed to be linear through the concrete section and the [illegible]m and minimum stresses are respectively -1.044 ksi and [illegible] wh[illegible] show[illegible] that no cracks will develop.

It m[illegible] the deflections for both types of lining have been ass[illegible] arbitrarily a[illegible] that the final stresses which are calculated give [illegible] a crude evaluation of the safety of the preliminary design.

The preliminary design data is now input into the program based on the method of Anders Bull, in order to reach a more proper design by the knowledge of the stresses induced by the original loading and the soil reactions.

8.3 Application of the method of Anders Bull and final design:

A 6x6WF25 Corten B steel rib at 4 ft. intervals has been selected for the temporary support and an arbitrarily chosen 12 in. concrete thickness is first tried for the permanent lining.

The rib spacing and the concrete thickness are selected so that the maximum compressive stress or the maximum tensile stress does not reach the maximum allowable stress divided by a factor of safety of 1.5 for instance. The maximum compressive stress varies linearly with the rib spacing as first shown in the parametrical study (Fig. 7.4) and only two points are necessary to determine the curve. For the concrete lining, the maximum stresses do not vary linearly with the concrete thickness, (Fig. 7.9) and a minimum of 3 points must be calculated to determine the curve. The cases of a temporary support with 8x6 WF28 rib with 4 ft. spacing and 8x8WF35 rib with 4 ft. spacing have also been studied. Results of the calculations are summarized in Table 8.1.

The results indicate a shortening of the horizontal diameter and a lengthening of the vertical diameter, because the vertical earth pressure is calculated according to Terzaghi's arching theory and the

TYPE OF LINING	MAX.DEFLECTION (FT.)	AV.THRUST (KIPS)	MAX.MOMENT (KIPSxFT.)	MAX.STRESS (KSI.)
RIB AND LAGGING - LOADING (1)				
6x6WF25 RIB WITH 4FT.SPACING	-0.0044	190.5	-5.1914	-29.5743
6x6WF25 RIB WITH 3FT.SPACING	-0.0038	142.6	-4.4854	-22.6839
8x6WF28 RIB WITH 4FT.SPACING	-0.0032	189.2	-6.8730	-26.2977
8x8WF35 RIB WITH 4FT.SPACING	-0.0028	188.9	-7.3928	-21.1171
				MAX.AND MIN.STRESS
CONCRETE LINING - LOADING (2)				(KSI.)
12 IN. THICK	-0.0045	143.8	+6.0530	-0.4858
			-5.6589	-1.4626
6 IN. THICK	-0.0087	143.7	+2.0162	-1.3318
			-2.2524	-2.7443
4 IN. THICK	-0.0168	144.3	1.6941	-1.7437
			-1.4334	-4.2842

LOADING (1): CALCULATED ACCORDING TO TERZAGHI'S ARCHING EFFECT THEORY AND CORRESPONDING TO A FULL OVERBURDEN PRESSURE UNDER 32.6 FT. OF SOIL.

LOADING (2): CORRESPONDING TO A FULL OVERBURDEN PRESSURE UNDER 115. FT. OF SOIL.

TABLE 8.1 - TUNNEL DESIGN BY USE OF THE METHOD OF ANDERS BULL - SUMMARY OF RESULTS.

horizontal earth pressure is calculated from the full overburden pressure with a K_0 value of 1. The calculated thrusts are found higher than those due to the overburden pressure only, because of the added effect of the soil reactions on the tunnel crown and invert. Finally, no tensile stresses develop in the concrete lining, so that the design of the concrete liner is based on the allowable compressive stress only.

For a factor of safety of 1.5, Fig. 8.1 determines a rib spacing of 2.6 ft., with 6x6WF25 ribs and a concrete thickness of 6 in., with CPA350 concrete. A rib spacing of 2.6 ft., would reduce the rate of advance of the construction, since the mole can be jacked forward 4 ft., at once. An alternative is to use either a 3 ft., rib spacing with 6x6WF25 ribs or a 4 ft., rib spacing with 8x8WF35 ribs, for which the factor of safety is respectively 1.3 and 1.4 (Table 8.1). With heavier ribs, the use of the mole is optimised since the rib spacing is more important but the required concrete thickness increases because of the technical limitations introduced by Edmonton construction procedures: the injection pipe for large tunnel concreting is an elliptic pipe 5.5 in., high and a clearance of 2 in., between pipe and ribs is judged necessary for facility of construction. To this total of 7.5 in., of concrete must be added the depth of the rib minus 2 in., when the 2 in., thick lagging is placed in between the ribs. The concrete thickness is then 15.6 in., or 12.7 in., for 8x8WF35 ribs and 13.6 in., or 10.7 in., for 6x6WF25 ribs. Since a concrete thickness of 6 in.,

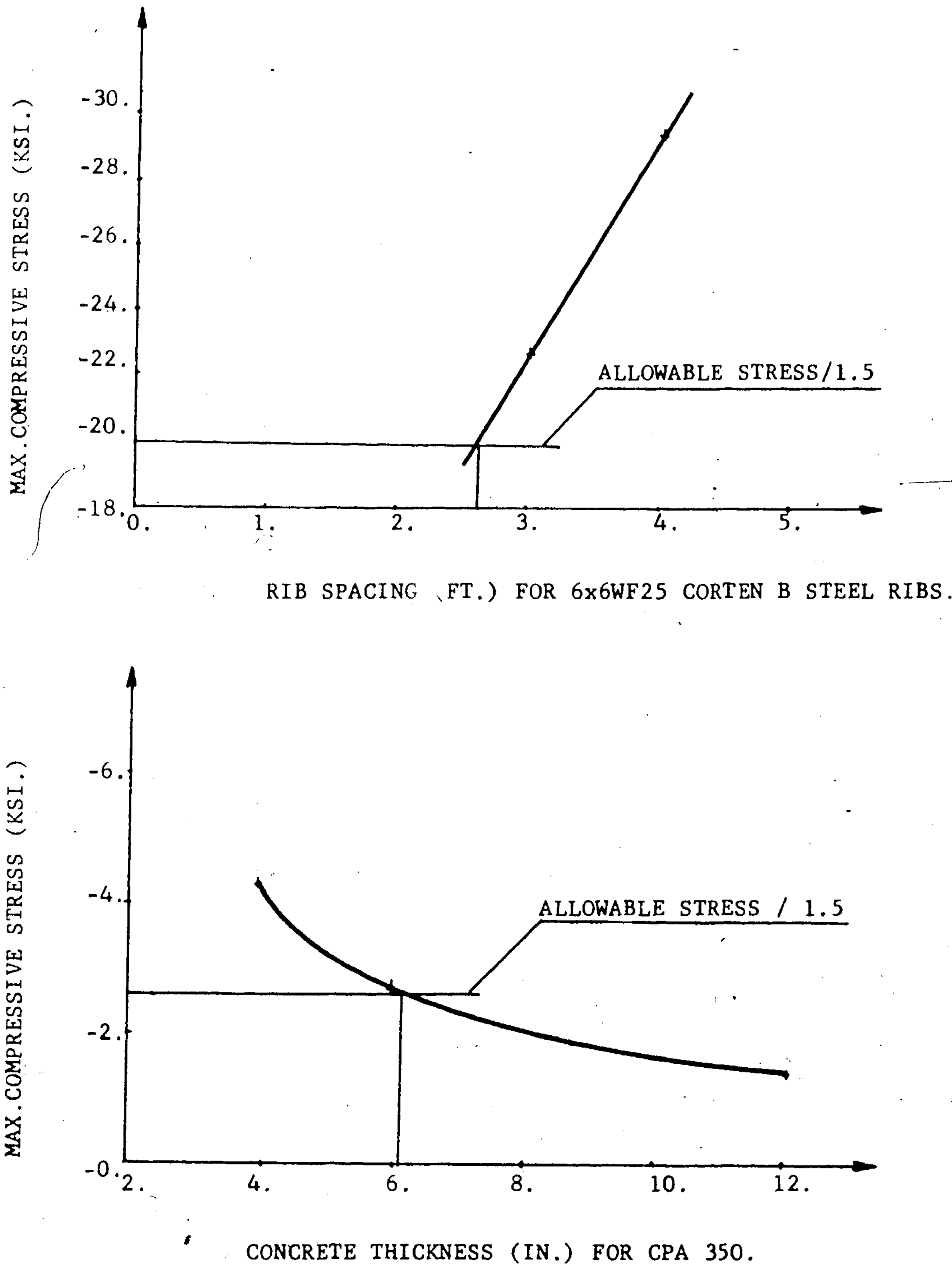


FIG.8.1 - TUNNEL DESIGN BY USE OF THE METHOD OF ANDERS BULL - GRAPHIC DETERMINATION.

only is required with a factor of safety of 1.5, the final concrete lining is designed only according to the technical limitations.

The final design must also be the most economic and consequently is a compromise between the cost of the required steel and concrete, the cost of the required work-hours and the optimisation of the use of the mole and the tunnel equipment. It is then proposed a 6x6WF25 rib with a 3 ft., spacing as temporary support and a 12 in., thick concrete lining.

For comparison, the City of Edmonton design services selected 6x6WF25 ribs with a 4 ft., spacing and the same thickness of concrete. Table 8.1 shows that for such a design, the factor of safety of the temporary support is 1.01. However, this conclusion is valid only if the assumptions in this study are proved correct by field measurements.

8.4 Concluding remarks:

The previous example of tunnel design illustrates the use of the method of Anders Bull. The K_0 value was assumed to be 1.0 and the soil reaction coefficient was taken as 200. units. The earth pressure acting on the temporary support was calculated according to Terzaghi's arching effect theory and the concrete lining was designed to resist the full overburden pressure by itself. The final design is valid as long as these assumptions can be checked satisfactorily by field data. If no data is available, a parametrical study similar to that described in Chapter 7 must be undertaken.

CHAPTER IX

Outline of a Field Measurement Program

9.1 Introduction:

The field measurement program must provide the data required for the calculations, and the measurements necessary to check these calculations.

The methods which are used (Bodrov-Gorelik, Anders Bull, Finite Element Method) require the knowledge of the physical properties of the material and of their elastic or non-elastic properties defined by the modulus of elasticity, the soil reaction coefficient or the ground reaction curve.

9.2 Physical properties:

Dejong (1971) gave a detailed description of the glacial till in Edmonton and a study of the variations of the modulus of elasticity with contact pressure. The physical properties of the Lake Edmonton Sediments have been described by S. Thomson (1969) and Sharma (1970).

9.3 Elastic-plastic properties:

In addition to the values of the modulus of elasticity given by Dejong for the glacial till, it is suggested that the soil reaction coefficient be determined at the depths of the tunnels for common Edmonton materials. No measurements of the elastic properties of the glacial till have been performed so far at great depths and some variations from the values given by Dejong might be found. The soil reaction coefficient can be measured by plate bearing tests; it is preferable to use 1 sq. ft., rigid circular plates mounted at the extremities of a frame which is expanded in its middle by a jacking system. The settlements of the plates are measured by extensometers. The measuring apparatus and the tests can follow the pattern developed by Sherif and Strazer (1973) for the determination of soil parameters in the design of Mt. Baker Ridge Tunnel in Seattle. The initial loading on the lining should be determined according to the recommendations of Deere et al (1969), prescribing the use of a ground reaction curve or earth pressure versus soil deformation curve.

This curve can be divided into four parts, (Fig. 2.1):

1) deformations due to the flow of the soil towards the working face.
2) deformations of the unlined tunnel.
3) deformations of the tunnel with the temporary lining.
4) deformations of the tunnel with the temporary and the permanent linings.

The deformations which are considered here apply to the points of the soil which are to bec~~ome or~~ are at the surface of the excavation.

9.4 Measurements of deflections:

The deformations of parts 1 and 2 can be determined with inclinometers installed very close to the section of the future excavation, on both sides of the tunnel axis. One must be aware, however, that there is no easy way of measuring the corresponding stress release at the same points, because they are still in the soil mass. This implies that the ground reaction curve can only be approximated for parts 1 and 2 according to the elastic theory or to the results of plate bearing tests, on the basis of the observed deformations. (See Deere, Peck et al 1969 f theoretical approximations).

The deformations of parts 3 and 4 can be measured from the inside of the tunnel; diameter changes can be measured between brass studs fixed on the lining with a steel tape or a telescopic rod. A minimum of 4 diameters per rib or section should be surveyed in order to give a reasonable picture of the lining deformations. The settlements of the crown or the invert should also be surveyed. These measurements should extend over a relevant period of time to show the time effect. Because of the many local irregularities, several consecutive ribs or sections should be observed.

9.5 Measurement of earth pressure:

The earth pressure distribution can be directly evaluated from earth pressure cells which are placed between lagging and ground. A flexible thin rectangular membrane can be inserted in the clearance and its volume variations recorded. The earth pressure distribution is reasonably approximated by eight cells per rib or section. It is preferable at first to instrument a tunnel which is subjected to a symmetrical loading; even in this case, experience proves that the observed symetric lateral pressures are not equal usually, so that the full perimeter must be instrumented. To allow for possible local disturbances, a minimum of three consecutive ribs or sections should be equipped.

One difficult point of the design is to determine the respective shares of the loads in the final configuration between temporary and permanent linings. The two linings however do not act perfectly independently since the ribs may be within the concrete mass, but the permanent lining cannot be considered reinforced either since the ribs can be only partially in the concrete. For simplicity of calculations, it is assumed that the two linings are separated. The distribution of the loads on the linings can only be backcalculated from the displacements of the final configuration of the two linings.

Samples of lagging from old tunnels should be tested for their strength and compared to the performance of new lagging. These samples can be obtained when a tunnel under construction breaks into an older part of the system. The tests would add important knowledge concerning the loads the concrete liner has to carry in the long term.

9.6 Measurements of lining stresses:

The previous data enables one to predict thrusts, moments and deflections of the linings. Another set of data must provide the stresses observed in-situ; the stresses in the lining can be measured by strain gauges which are installed on the inner and outer flange and on the neutral axis of the ribs. Due attention should be paid to the measurement of bending moments which do not act in the plane of the rib section. Strain gauge configuration and the general test procedure can be similar to the pattern described by Burke (1957) for the Garrison Dam Tunnels. Extension gauges are used in the concrete.

For better understanding of the deformations, it is useful to record the radial displacements of the lagging when jacks act upon it to push the mole forward.

If the lining is designed according to safety considerations only, the previous field measurement program is sufficient. However, the design of the lining involves also the considerations of the surface effects; surface settlements should be observed along 3 lines perpendicular to the tunnel axis, while a set of piezometers can be used to determine a possible drainage effect of the tunnel.

This field measurement program would enable one to predict the behaviour of a tunnel at any time and to verify the validity of the analysis with the measured field values.

In the eventuality that new lining materials or new constructi procedures are selected, it is recommended that a test section be built and instrumented as described previously.

CHAPTER X

Conclusions

This study presents a method of calculation of the moments, thrusts and deflections of a lining under a given symmetric loading in an elastic homogeneous medium.

The calculations are based upon the coefficient of ground reaction which must be known only approximately and on the magnitude of the arching effect, which must be known precisely.

The critical factors governing the behaviour of a lining are the rib spacing for a rib-and-lagging lining and the concrete thickness for the permanent lining. These parameters are evaluated by trial and error with the help of the method of Anders Bull.

It has been found that the predicted deflections are smaller than the measured ones, and that the construction procedures contribute to this discrepancy by an unknown amount.

Because of lack of field data to check the computed stresses, a field measurement program is proposed.

Further developments can be undertaken: the method can be written for an unsymmetric loading and can be applied to stratified homogeneous elastic materials. It can also be extended to deal with homogeneous elasto-plastic materials, for which the ground reaction curve is approximated by straight lines, each of these sections being considered as elastic for the calculations.

If the long term reliability of the temporary support is demonstrated, further studies would have to solve the problem for double-lined tunnels.

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APPENDIX I

Some details of the analysis of Anders Bull

I.1 Literal expressions of A_T, A_V, A_m:

Coefficients relative to radial loads:

$$A_{T_r} = - \frac{\alpha \sin\alpha}{2\pi}$$

$$A_{v_r} = - \frac{\sin\alpha - \alpha\cos\alpha}{2\pi}$$

$$A_{m_r} = - \frac{\alpha\sin\alpha + 1 - \cos\alpha}{2\pi}$$

Coefficients relative to tangential loads:

$$A_{T_t} = \mp \frac{\sin\alpha - \alpha\cos\alpha}{2\pi}$$

$$A_{v_t} = \mp \frac{2(1-\cos\alpha) - \alpha\sin\alpha}{2\pi}$$

$$A_{m_t} = \mp \frac{\alpha(1-\cos\alpha)}{2\pi}$$

with positive sign when α is between 0 and π, and negative sign when α is between π and 2π.

I.2 Correction for the moments:

The pressure line due to a uniform load is a circle coinciding with the neutral fibers of the lining. (Fig. 1 of Appendix I). Approximating the circular lining by a polygon introduces moments at each joint that have the following value:

$$M = + 0.01644 \times P_R \times R$$

where R is the radius of the neutral circle. Since the moments at the joints should be null under a uniform pressure, a moment of equal value but opposite sign must be added at each joint. The value P_R is taken as the average of the radial forces P'_R and P''_R acting at the mid-joint of the segments on both sides of the joint under study:

$$- M = - 0.01644 \times \frac{P'_R + P''_R}{2}$$

No adjustment of the coefficients is required for the evaluation of the thrusts and the shear reactions, since the approximation of the lining by a polygon does not induce any consequent difference for these quantities in the results.

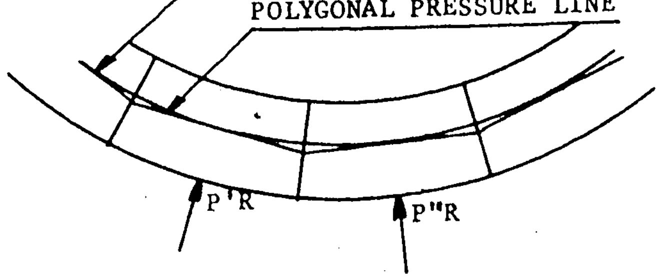


FIG.I.1 - CORRECTION FOR MOMENT COEFFICIENTS.

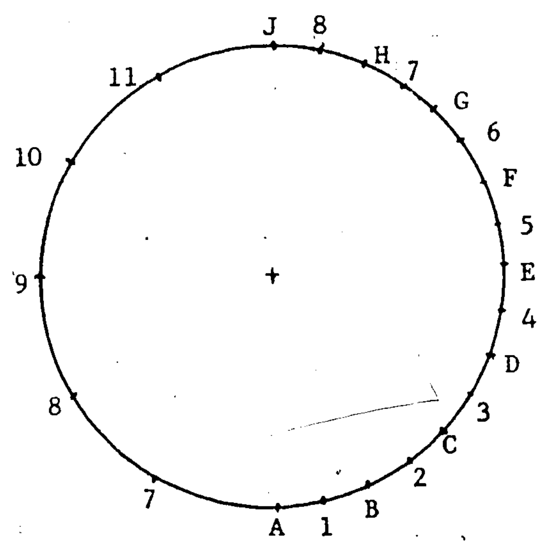


FIG.I.2 - POINTS AND JOINTS POSITIONS.

NUMBERING OF JOINTS :								
BY ANDERS BULL - A	B	C	D	E	F	G	H	J
IN PROGRAM - 1	2	3	4	5	6	7	8	9
CLOCK - 6				3,9				12

APPENDIX II

Listings of the computer program and of input and output data

II.1 Reading order of data:

The symbols which are used are defined in Table 3.1 of the text.

The three options of loading definition are considered.

- If the radial and tangential forces are given:

  1) 00
  2) PR(I), I=1,8
  3) PT(I), I=1,8
  4) N, R, AK, AI, E, RO
  5) SPACNG
  6) AS, A
  7) NZ(I), I=1,8
  8) Coefficients

- If the vertical and horizontal stresses are given:

  1) 02
  2) σ_V (I), I=1,8
  3) σ_H (I), I=1,8
  4) N, R, AK, AI, E, RO
  5) SPACNG
  6) PW

7) AS, A

8) NZ (I), I=1,8

9) Coefficients

- If the subroutine LOADING is used:

1) 01

2) KODE, GMA, GMS, GMW, SK, DEPTH, HW, PW, RO, SPACNG

3) N, R, AK, AI, E

4) AS, A

5) NZ(I), I=1,8

6) Coefficients

II.2.- LISTING OF INPUT AND OUTPUT FILES FOR THE NUMERICAL EXAMPLE OF ANDERS BULL.

```
$list buldat
>     2    00
>     3      08.966  10.388  12.144  12.920  12.505  13.102  14.290  15.159
>     4      -0.147  -0.418  -0.625  -0.738  -0.738  -0.625  -0.418  -0.147
>     5      5102.1  12.  136.              12000.    8.71
>     6     1.
>     7     23.427.6
>     8    1 2 3 4 5 0 0 0
>     9    -0.00215-0.00657-0.00753-0.00540-0.00115+0.00384+0.00816+0.01065
>    10    -0.00657-0.03747-0.05142-0.04068-0.01252+0.02266+0.05385+0.07200
>    11    -0.00753-0.05142-0.09098-0.08340-0.03397+0.03455+0.09766+0.13497
>    12    -0.00540-0.04068-0.08340-0.09611-0.05254+0.02546+0.10276+0.14980
>    13    -0.00115-0.01252-0.03397-0.05254-0.04653-0.00163+0.05536+0.09291
>    14    +0.00384+0.02266+0.03455+0.02546-0.00163-0.02736-0.03121-0.02635
>    15    +0.00816+0.05385+0.09766+0.10276+0.05536-0.03121-0.11947-0.16714
>    16    +0.01065+0.07200+0.13497+0.14980+0.09291-0.02635-0.16714-0.20686
>    17    -0.00016-0.00199-0.00487-0.00749-0.00883-0.00839-0.00590-0.00213
>    18    -0.00045-0.00896-0.02731-0.04611-0.05696-0.05502-0.03970-0.01445
>    19    -0.00051-0.01135-0.04034-0.07626-0.10033-0.10043-0.07392-0.02715
>    20    -0.00036-0.00862-0.03332-0.07918-0.10106-0.10693-0.08119-0.03024
>    21    -0.00007-0.00234-0.01128-0.02868-0.04934-0.05979-0.04999-0.01891
>    22    +0.00026+0.00531+0.01709+0.02962+0.03466+0.02846+0.01634+0.00505
>    23    +0.00055+0.01201+0.04154+0.08354+0.11625+0.12179+0.09146+0.03359
>    24    +0.00072+0.01599+0.05742+0.11549+0.16554+0.18015+0.14193+0.05423
>    25     0.18290 0.45140 0.57160 0.55170 0.42919 0.25980 0.10420 0.01220
>    26     0.69259 0.69559 0.71420 0.57500 0.33110 0.05490-0.18220-0.31710
>    27     1.20500 0.95520 0.69880 0.49360 0.20000-0.11090-0.36730-0.51150
>    28     1.46780 1.20840 0.73460 0.32380 0.05160-0.22140-0.48920-0.56170
>    29     1.47120 1.24720 0.83340 0.20260-0.09750-0.27780-0.41570-0.49040
>    30     1.25060 1.09610 0.80520 0.41200-0.03670-0.29190-0.32790-0.74450
>    31     0.87550 0.80860 0.67480 0.47590 0.21770-0.08690-0.22060-0.18200
>    32     0.43350 0.45420 0.47920 0.48040 0.42570 0.28890 0.05910-0.05820
>    33     0.0122   0.1042   0.2598   0.4291   0.5517   0.5716   0.4514   0.1829
>    34    -0.9816  -0.8524  -0.6466  -0.4219  -0.2268  -0.0910  -0.0209  -0.0008
>    35     0.1392  -0.6015  -0.7190  -0.0614   0.1128   0.1943   0.1667   0.0646
>    36     0.2408   0.0727  -0.0163   0.2218   0.3597   0.3766   0.2798   0.1029
>    37     0.2926   0.8261   1.2134   0.4049   0.4795   0.4453   0.7129   0.1123
>    38     0.2926   0.8334   1.2472   1.4712   0.4904   0.4157   0.2778   0.0975
>    39     0.2481   0.7138   1.0911   1.3327   1.4074   0.3229   0.2005   0.0679
>    40     0.1730   0.5059   0.7993   1.0273   1.1652   1.1922   0.1130   0.0351
>    41     0.0847   0.2585   0.4421   0.6318   0.8121   0.9554   1.0267   0.0101
>    42     0.0008   0.0209   0.0910   0.2268   0.4219   0.6466   0.8524   0.9816
>    43     0.1603   0.3978   0.4302   0.2955   0.0487  -0.2353  -0.4788  -0.6183
>    44     0.6544   0.6134   0.5592   0.3141  -0.0446  -0.4277  -0.7450  -0.9239
>    45     1.1075   0.8158   0.4974   0.2191  -0.1603  -0.5543  -0.8727  -1.0506
>    46     1.2690   0.9914   0.4758   0.0210  -0.2905  -0.6074  -0.8598  -0.9994
>    47     1.1529   0.9289   0.5150  -0.0339  -0.4241  -0.5961  -0.7349  -0.8087
>    48     0.8128   0.6765   0.4192   0.0700  -0.3395  -0.5508  -0.6450  -0.5433
>    49     0.3365   0.3031   0.2314   0.1136  -0.0567  -0.2883  -0.7600  -0.2796
>    50    -0.1732  -0.1086  -0.0025   0.1047   0.1648   0.1340  -0.0229  -0.0063
>    51    -0.6183  -0.4788  -0.2353   0.0487   0.2955   0.4302   0.3978   0.1603
>    52     0.0180   0.1369   0.3055   0.4528   0.5229   0.4861   0.3434   0.1238
>    53     0.1341   0.3744   0.6130   0.7895   0.8468   0.7513   0.5176   0.1845
>    54     0.2223   0.6102   0.8584   1.0050   1.0180   0.8762   0.5924   0.2093
>    55     0.2538   0.7062   1.0022   1.0868   1.0365   0.8591   0.5680   0.1986
>    56     0.2301   0.6459   0.9347   1.0337   0.9279   0.7282   0.4653   0.1600
>    57     0.1619   0.4586   0.6773   0.7758   0.7254   0.5341   0.3203   0.1066
>    58     0.0666   0.1933   0.2997   0.3690   0.3820   0.3175   0.1755   0.0536
>    59    -0.0351  -0.0924  -0.1149  -0.0941  -0.0389   0.0233   0.0508   0.0152
>    60    -0.1238  -0.3434  -0.4861  -0.5229  -0.4528  -0.3055  -0.1369  -0.0180
#END OF FILE
```

$run bulcom+*ssplib+*plotlib 5=buldat 9=-f 3=loares
#22:19.41
IMPOSED LOADING :

I	PR(I)	PT(I)
1	8.966	-0.147
2	10.388	-0.418
3	12.144	-0.625
4	12.920	-0.738
5	12.505	-0.738
6	13.102	-0.625
7	14.290	-0.418
8	15.159	-0.147

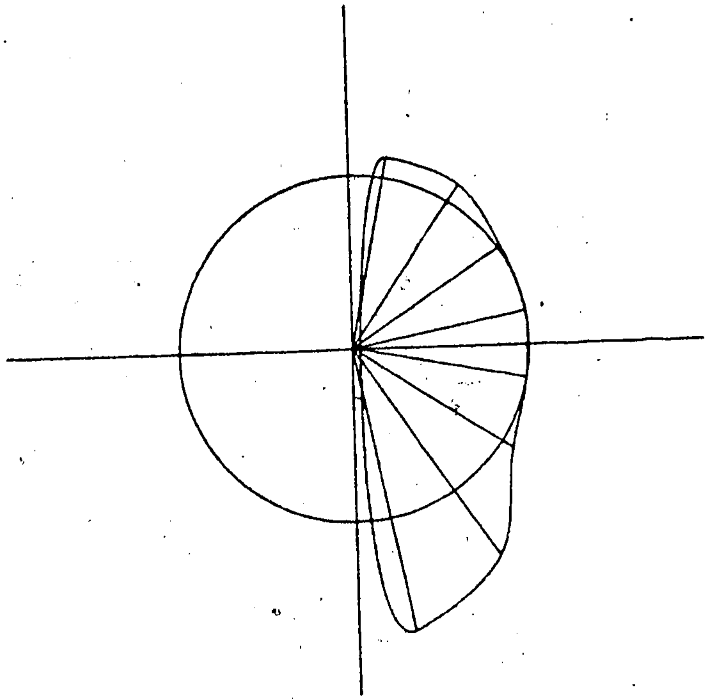
N	R	AK	AI	E	AS	A
5	102.100	12.000	136.000	12000.0	23.400	27.600

I	R(I)	AD(I)	AT(I)	AM(I)	SIGO(I)	SIGI(I)
1	6.1555	0.0126	39.9618	0.4506	-1.2168	-1.6790
2	5.1525	0.0105	39.7276	-0.3609	-1.6245	-1.2543
3	3.8628	0.0079	39.1580	-1.4659	-2.1705	-0.6670
4	2.7482	0.0056	38.5801	-1.2802	-2.0543	-0.7413
5	1.1526	0.0024	38.1989	1.8925	-0.4135	-2.3545
6	0.0	-0.0045	37.7206	4.2828	0.8296	-3.5630
7	0.0	-0.0140	36.8341	2.0100	-0.3038	-2.3653
8	0.0	-0.0210	35.8823	-2.6478	-2.6579	0.0577
9	-0.0000	-0.0000	35.4834	-4.8538	-3.7748	1.2035

STOP 0
#22:21.32 2.791 RC=0
#

II.3.- LISTING OF INPUT AND OUTPUT FILES, AND COMPUTER PLOTS FOR A RIB AND LAGGING LINING UNDER A SYMMETRICAL LOADING. (6x6WF25 CORTEN B STEEL RIBS AT 4FT. INTERVAL).

```
 $run bulcom**ssplib**plotlib 5=buldtl 9=-f 3=loares
#22:00.53

                    STEEL          SPACING =    4.000 FT.

 LOADING = FULL OVERBURDEN PRESSURE

   GMA      GMS      GMW      SK      DEPTH  HW        PW       PO
   125.000150.000 63.400   1.000 32.600200.000 91.037   9.530

           I    PR(I)    PT(I)
           1     3.577  -0.018
           2    28.834  -0.051
           3    61.300  -0.076
           4    79.162  -0.089
           5    75.373  -0.089
           6    68.977  -0.076
           7    64.082  -0.051
           8    61.433  -0.018

     N        R        AK        AI        E         AS        A
     6        111.360  200.000   53.500  30000.0   16.800    7.370

   I      B(I)      AD(I)      AT(I)      AM(I)     SIGO(I)   SIGI(I)

   1    70.6311    0.0020   191.4126    0.6028   -25.5413  -26.4024
   2    48.3243    0.0014   191.3092   -0.6168   -26.3984  -25.5173
   3     7.6439    0.0002   191.5652    4.0745   -23.0822  -28.9029
   4     0.0      -0.0044   191.1816   -0.1922   -26.0778  -25.8032
   5     0.0      -0.0036   190.6187   -5.1943   -29.5743  -22.1539
   6     1.9341    0.0001   191.1709    1.3843   -24.9503  -26.9278
   7    12.3947    0.0003   191.1867    2.1343   -24.4167  -27.4657
   8    12.7320    0.0004   190.8521   -0.6060   -26.3287  -25.4629
   9    -0.0000   -0.0000   190.9046    0.2182   -25.7470  -26.0588

  STOP      0
#22:03.03 2.985 RC=0
#$run *calcompq scards=-f
#22:03.23
 *** CALCOMP RECEIPT #   2360****   PLOT LENGTH =  14 INCHES;   PLOT TIME=   5 I
#22:03.28 1.131 RC=0
#$list buldtl(1,6)
>      1      01
>      2      1  125.    150.     63.4     1.      32.6  200.     25.     9.53  4.
>      3        6111.36200.      53.5              30000.
>      4       16.8 7.37
>      5      1 2 3 6 7 8 0 0
>      6      -0.00215-0.00657-0.00753-0.00540-0.00115+0.00384+0.00816+0.01065
#END OF FILE
#
```



MAX. OF ABSOLUTE VALUE = 1.5 CM.
OUTPUT VALUES IN ANNEX II.3
UNITS IN TABLE 3.1.

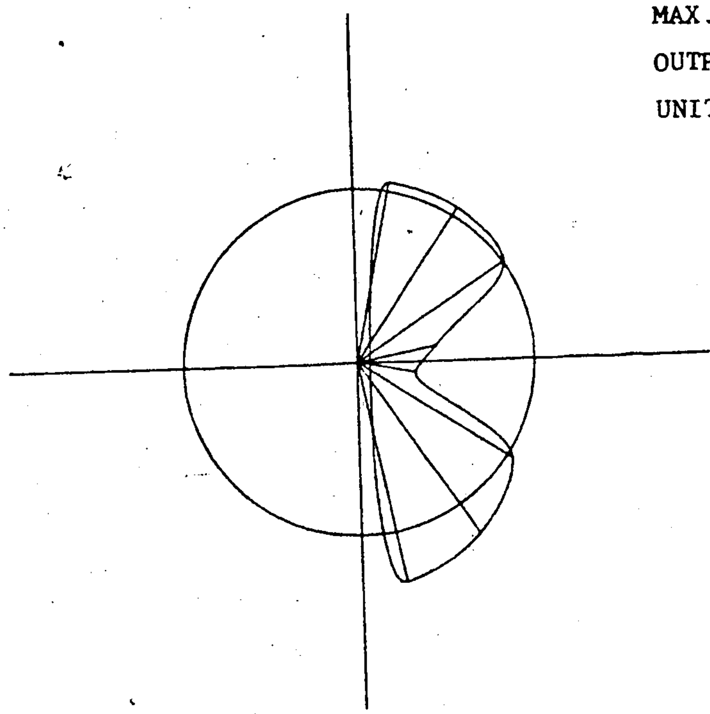


FIG.II.1- DISPLACEMENTS AND SOIL REACTIONS COMPUTER PLOTS FOR 6x6WF25 CORTEN B STEEL RIBS AT 4FT. SPACING UNDER SYMMETRICAL LOADING.

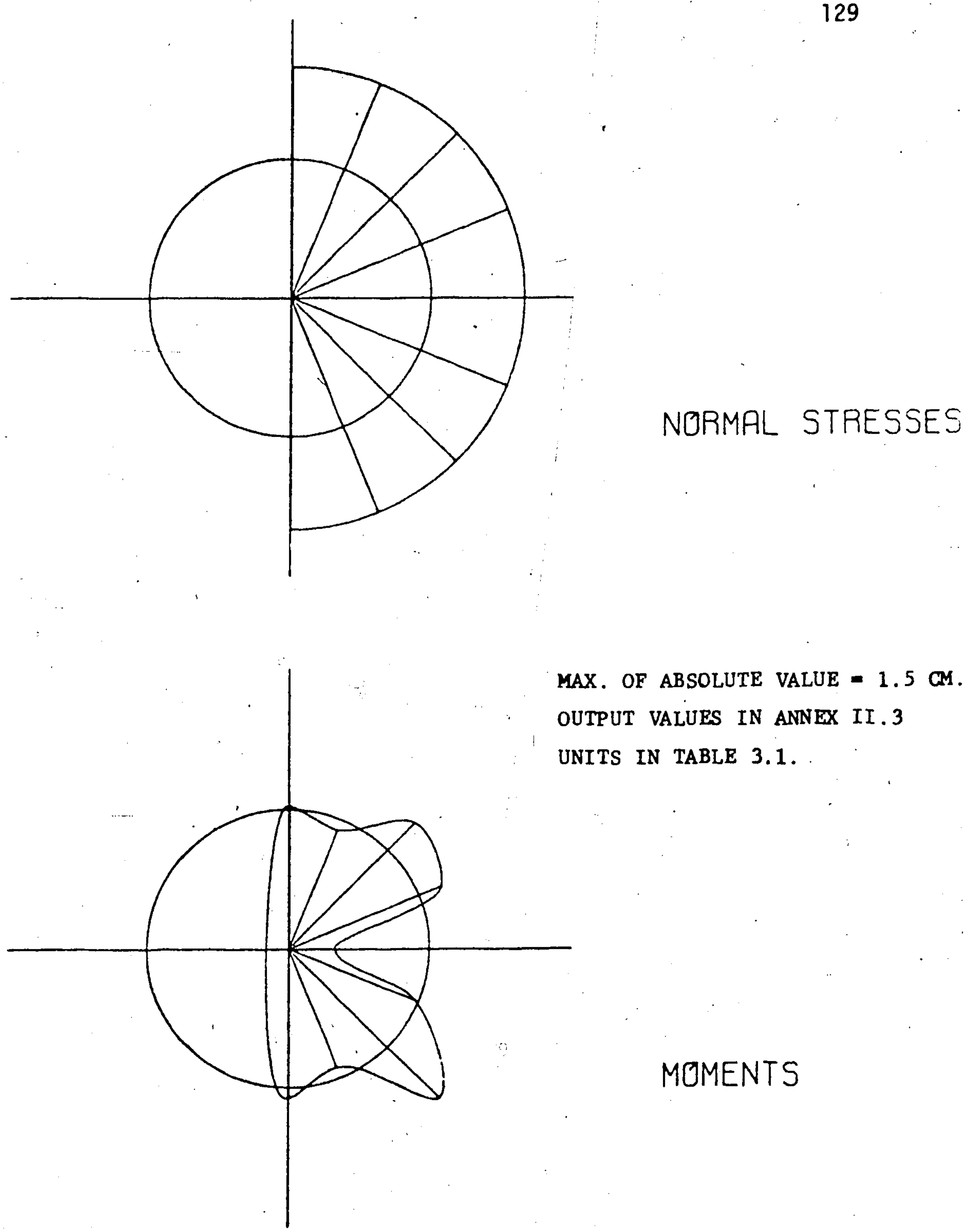


FIG.II.2- MOMENTS AND THRUSTS COMPUTER PLOTS FOR 6x6WF25 CORTEN B STEEL RIBS AT 4FT. SPACING UNDER SYMMETRICAL LOADING.

II.4 - LISTING OF THE PROGRAM BASED ON ANDERS BULL METHOD.

```
      C
      C
      C DIMENSION OF B,C,DRMD,AB,AC,L,M,ABM,ACM MUST BE N OR (N,N) OR (N,1)
      C THE VALUE OF THESE DIMENSIONS MUST BE CHANGED WHEN N IS CHANGED .
      C
      C THE PR(I) AND PT(I) VALUES MUST BE ENTERED THE LOWEST POINT FIRST.
      C
            DIMENSION DR(8,8),DT(8,8),DRM(8,9),AF(8),CO(8),SI(8)
            DIMENSION DRMDD(100)
            DIMENSION B(10),C(10),DRMD(100),AB(10,1),AC(10,1)
            DIMENSION L(10),M(10),ABM(10,1),ACM(10,1)
            DIMENSION AD(9),BTR(8,9),BTI(8,9),BMR(8,9),BMI(8,9),AT(9),AM(9)
            DIMENSION SIGO(9),SIG1(9)
            DIMENSION BF(9),NZ(8),BZ(8)
            COMMON H(8),AH(8),AV(8),BTA(8),PR(8),PT(8)
            COMMON KODE,GMA,GMS,GMW,SK,DEPTH,HW,PW,RO,SPACNG
            DIMENSION DATA(2000)
      C
      C N IS THE NUMBER OF SEGMENTS ON WHICH WE ASSUME SOIL REACTIONS WILL
      C     DEVELOP
      C
            READ(5,600)KODLD
            IF(KODLD-1)7,1,8
      C
      C LOADING DEFINED BY RADIAL AND TANGENTIAL LOADS
      C
          7 READ(5,610)PR,PT
            READ(5,620)N,R,AK,AI,E,RO
            READ(5,622)SPACNG
            WRITE(6,2050)
            GO TO 5
      C
      C
      C LOADING DEFINED BY HORIZONTAL AND VERTICAL STRESSES
          8 READ(5,610)PR,PT
            READ(5,620)N,R,AK,AI,E,RO
            READ(5,622)SPACNG
            READ(5,622)PW
            WRITE(6,2050)
            WRITE(6,3100)SPACNG,PW
            PW=PW*.0327*R
            CALL LOAD2
            GO TO 5
      C
      C LOADING AUTOMATICALLY CALCULATED FOR FULL OVERBURDEN PRESSURE
      C
          1 READ(5,700)KODE,GMA,GMS,GMW,SK,DEPTH,HW,PW,RO,SPACNG
            READ(5,900)N,R,AK,AI,E
            IF(KODE-1)2,3,3
          2 WRITE(6,702)
            SPACNG=1.
            GO TO 4
          3 WRITE(6,703)SPACNG
          4 CONTINUE
            PW=PW*.0327*R
            CALL LOADNG
            WRITE(6,2060)
            WRITE(6,720)
            WRITE(6,730)GMA,GMS,GMW,SK,DEPTH,HW,PW,RO
            GO TO 5
```

```
C
    5 WRITE(6,2065)
      DO 6 I=1,8
    6 WRITE(6,2070)I,PR(I),PT(I)
      READ(5,800)AS,A
      READ(5,2090)NZ
      READ(5,1100)((DR(I,J),I=1,8),J=1,8),((DT(I,J),I=1,8),J=1,8)
      READ(5,1105)BTR,BTT
      READ(5,1105)BMR,BMT
      S=2.*RO*SIN(3.1416/16.)*SPACNG
C
C CALCULATION OF DEFLECTIONS, THRUSTS AND MOMENTS
C
      Q=E*AI/((R**3.)*AK*S)
      DO 20 I=1,8
      DO 10 J=1,8
   10 DRM(I,J)=DR(I,J)
   20 DRM(I,I)=DRM(I,I)-Q
C
C CALCULATING CO AND SI
C
      AF(1)=3.1416/16.
      DO 30 I=1,7
   30 AF(I+1)=AF(I)+3.1416/8.
      DO 40 I=1,8
      CO(I)=COS(AF(I))
   40 SI(I)=SIN(AF(I))
      BF(1)=0.
      DO 45 I=1,8
      J=I+1
   45 BF(J)=AF(I)+AF(I)
C
C CALCULATING B IN : DRMD*F=B*DELT+C
C
      AQ=-E*AI/(R**3.)
      DO 50 I=1,8
   50 B(I)=AQ*CO(I)
C
C CALCULATING C
C
      DO 60 I=1,8
      C(I)=0.
      DO 60 J=1,8
   60 C(I)=C(I)-PR(J)*DR(J,I)-PT(J)*DT(J,I)
C
C RESTRICTING DRM(8,8) TO DRMD(N,N)
C
      J1=1
      DO 70 J=1,N
      JZ=NZ(J)
      DO 70 I=1,N
      IZ=NZ(I)
      DRMDD(J1)=DRM(IZ,JZ)
      DRMD(J1)=DRM(IZ,JZ)
   70 J1=J1+1
C
C CALCULATING THE INVERSE MATRIX OF DRMD.
C
      CALL MINV(DRMD,N,D,L,M)
C
```

```
C CALCULATING THE PRODUCTS DRMO-1*B AND DRMD-1*C :
C
      DO 80 I=1,N
      IZ=NZ(I)
      AB(I,1)=B(IZ)
   80 AC(I,1)=C(IZ)
      CALL GMPRD(DRMD,AB,ABM,N,N,1)
      CALL GMPRD(DRMD,AC,ACM,N,N,1)
C
C CALCULATING C2 AND C3 :
C
      C2=0.
      C3=0.
      DO 100 I=1,N
      IZ=NZ(I)
      C2=C2+CO(IZ)*ABM(I,1)
  100 C3=C3+CO(IZ)*ACM(I,1)
C
C CALCULATING C1
C
      C1=0.
      DO 110 I=1,8
  110 C1=C1-CO(I)*PR(I)-SI(I)*PT(I)
C
C CALCULATING DELTA
C
      DELT=(C1-C3)/C2
C
C CALCULATING THE RIGHT-HAND SIDE OF THE N EQUATIONS
C
      DO 120 I=1,N
      IZ=NZ(I)
      B(IZ)=DELT*B(IZ)+C(IZ)
  120 BZ(I)=B(IZ)
C
C SOLVING THE N SIMULTANEOUS EQUATIONS :
C
      CALL SIMQ(DRMDO,BZ,N,IK)
      DO 124 I=1,8
  124 B(I)=0.
      DO 125 I=1,N
      IZ=NZ(I)
  125 B(IZ)=BZ(I)
C
C RADIAL GROUND PRESSURE RESULTANT
C
      DO 180 I=1,N
      IZ=NZ(I)
  180 PR(IZ)=PR(IZ)+B(IZ)
C
C CALCULATING THE DISPLACEMENTS
C
      DO 140 I=1,8
      C(I)=0.
      DO 135 J=1,N
  135 C(I)=C(I)+PR(J)*DR(J,I)+PT(J)*DT(J,I)
      AD(I)=(R**3.)*C(I)/(E*AI)+DELT*CO(I)
  140 CONTINUE
      DO 145 I=1,8
  145 AD(I)=AD(I)/12.
```

```
      C
      C CALCULATING THRUSTS AND MOMENTS
      C
            DO 190 I=1,9
            AT(I)=0.
            AM(I)=0.
            DO 185 J=1,8
            AT(I)=AT(I)+PR(J)*BTR(J,I)+PT(J)*BTT(J,I)
        185 AM(I)=AM(I)+PR(J)*BMR(J,I)+PT(J)*BMT(J,I)
        190 AM(I)=R*AM(I)/12.
      C
      C CALCULATING STRESSES IN THE LINING
      C
            DO 220 I=1,9
            ST=ABS(AT(I)/A)
            SM=ABS(AM(I)*12/AS)
            IF(AM(I))200,200,210
        200 SIGO(I)=-ST-SM
            SIGI(I)=-ST+SM
            GO TO 220
        210 SIGO(I)=-ST+SM
            SIGI(I)=-ST-SM
        220 CONTINUE
      C
      C OUTPUT OF RESULTS
      C
            WRITE(6,2030)
            WRITE(6,2040)N,R,AK,AI,E,AS,A
            WRITE(6,2010)
            DO 230 I=1,9
        230 WRITE(6,2020)I,B(I),AD(I),AT(I),AM(I),SIGO(I),SIGI(I)
      C
      C SCALE FACTORS
      C
            CALL SCALE(AD,8,DD)
            CALL SCALE(B,N,DB)
            CALL SCALE(AM,9,DM)
            CALL SCALE(AT,9,DT)
      C
      C PLOTTING
      C
            CALL PLOTS(DATA(1),8000)
            CALL PLOT(0.0,0.0,3)
            CALL PLOTTR(3.5,3.5,DD,8,AD,AF)
            CALL PLOTTR(3.5,10.5,DB,8,B,AF)
            CALL PLOTTR(3.5,17.5,DM,9,AM,BF)
            CALL PLOTTR(3.5,24.5,DT,9,AT,BF)
      C
            CALL SYMBOL(7.5,2.0,0.25,13HDISPLACEMENTS,0.0,13)
            CALL SYMBOL(7.5,9.0,0.25,14HSOIL REACTIONS,0.0,14)
            CALL SYMBOL(7.5,16.0,0.25,7HMOMENTS,0.0,7)
            CALL SYMBOL(7.5,23.0,0.25,15HNORMAL STRESSES,0.0,15)
            CALL PLOT(0.0,0.0,999)
      C
        600 FORMAT(I2)
        610 FORMAT(8F8.3/8F8.3)
        620 FORMAT(I3,F6.2,F6.1,F6.2,F20.2,F6.3)
        622 FORMAT(F6.3)
        700 FORMAT(I1,2X,9F7.3)
        702 FORMAT(/20X,8HCONCRETE/)
```

```
  703 FORMAT(/20X,5HSTEEL,10X,10HSPACING = ,F7.3,4H FT./)
  720 FORMAT(3X,3HGMA,4X,3HGMS,4X,3HGMW,4X,2HSK,5X,5HDEPTH,2X,
     12HHW,5X,2HPW,5X,2HRO)
  730 FORMAT(3X,9F7.3/)
  800 FORMAT(2F5.1)
  900 FORMAT(I3,F6.2,F6.1,F6.2,F20.2)
 1000 FORMAT(8F8.3,/8F8.3)
 1100 FORMAT(8(8F8.5/),7(8F8.5/),8F8.5)
 1010 FORMAT(3X,8F8.3,/3X,6F8.3)
 1110 FORMAT(3X,8(8F8.5/3X),/8(8F8.5/3X))
 1200 FORMAT(/3X,5(5F8.5/),/3X,5F8.5,/3X,5F8.5)
 1305 FORMAT(3X,8F8.5)
 1400 FORMAT(/3X,53HNUMBER OF SEGMENTS ON WHICH SOIL REACTIONS DEVELOI
     1 ,I3/)
 1105 FORMAT(9(8F8.5/),8(8F8.5/),8F8.5)
 2010 FORMAT(2X,1HI,5X,4HB(I),6X,5HAD(I),6X,5HAT(I),6X,5HAM(I),6X,
     17HSIGO(I),4X,7HSIGI(I)/)
 2020 FORMAT(I3,F11.4,1X,5(E10.4,1X))
 2030 FORMAT(/5X,1HN,6X,1HR,8X,2HAK,7X,2HAI,7X,1HE,8X,2HAS,7X,1HA)
 2040 FORMAT(4X,I2,6X,3F9.3,F9.1,2F9.3/)
 2050 FORMAT(1X,17HIMPOSED LOADING :/)
 2060 FORMAT(1X,34HLOADING = FULL OVERBURDEN PRESSURE/)
 2065 FORMAT(/10X,1HI,2X,5HPR(I),3X,5HPT(I))
 2070 FORMAT(10X,I1,1X,2F8.3)
 2080 FORMAT(8(I1,1X))
 3100 FORMAT(/10X,9HSPACNG = ,F8.3,2X,5HPW = ,F8.3)
      STOP
      END
C
      SUBROUTINE SCALE(A,K,D)
      DIMENSION A(K)
      AMAX=ABS(A(1))
      MK=K-1
      DO 30 I=1,MK
      X=ABS(A(I+1))
      IF(X-AMAX)10,10,20
   10 GO TO 30
   20 AMAX=X
      GO TO 30
   30 D=1./AMAX
      RETURN
      END

      SUBROUTINE PLOTTR(XO,YO,D,K,A,AF)
      DIMENSION A(K),AF(K)
C
C CHANGING THE ORIGIN TO THE CENTER OF THE GRAPH :
C
      CALL PLOT(XO,YO,-3)
C
C DRAWING THE AXES :
C
      CALL PLOT(3.0,0.0,3)
      CALL PLOT(-3.0,0.0,2)
      CALL PLOT(0.0,3.0,3)
      CALL PLOT(0.0,-3.0,2)
C
C DRAWING THE CIRCULAR TUNNEL LINING :
C
      CALL CIRCLE(1.5,0.0,0.,360.,1.5,1.5,0.0
```

```
      CALL SMOOTH(0.0,0.0,0)
      DO 60 I=1,K
      A(I)=1.5+D*A(I)
      FI=AF(I)
      X=A(I)*SIN(FI)
      Y=-A(I)*COS(FI)
      IF(I-1)10,10,20
   10 CALL SMOOTH(X,Y,-1)
      GO TO 50
   20 IF(I-K)30,40,40
   30 CALL SMOOTH(X,Y,-2)
      GO TO 50
   40 CALL SMOOTH(X,Y,-25)
      GO TO 50
   50 CALL PLOT(0.0,0.0,3)
      CALL PLOT(X,Y,2)
   60 CONTINUE
      X1=-X0
      Y1=-Y0
      CALL PLOT(X1,Y1,-3)
      RETURN
      END
C
      SUBROUTINE LOADNG
C
      COMMON H(8),AH(8),AV(8),BTA(8),PR(8),PT(8)
      COMMON KODE,GMA,GMS,GMW,SK,DEPTH,HW,PW,RO,SPACNG
C
C GMA IS IN PCF, PW IN LB
C
      A=3.1416*RO/8.
      DO 1 I=1,8
      SI=I-1
      BTA(I)=3.1416*(.5+SI)/8.
      H(I)=DEPTH+RO+RO*COS(BTA(I))
      AH(I)=ABS(A*COS(BTA(I)))
    1 AV(I)=ABS(A*SIN(BTA(I)))
      WRITE(3,1100)H,AH,AV,BTA
      WRITE(3,1150)
      WRITE(3,1160)
C
      DO 50 I=1,8
      SIGV=GMA*H(I)
      SIGH=SK*SIGV
      IF(H(I)-HW)6,6,5
    5 SIGV=GMA*HW+GMS*(H(I)-HW)
      SIGH=SK*SIGV+GMW*(H(I)-HW)*(1.-SK)
    6 CONTINUE
      PV=SIGV*AH(I)*SPACNG+PW
      PH=SIGH*AV(I)*SPACNG
      IF(I-4)9,9,20
    9 IF(H(I)-HW)13,13,10
   10 PV=PW-GMW*(H(I)-HW)*SPACNG*AH(I)
      IF(PV)11,11,12
   11 PV=ABS(PV)
      BTA(I)=-BTA(I)
   12 GO TO 20
   13 PV=PW
   20 CONTINUE
      P=SQRT(PV**2.+PH**2.)
```

```
          ALPH=ATAN2(PH,PV)
          TETA=ALPH+BTA(I)
          PR(I)=ABS(P*COS(TETA))/1000.
          PT(I)=-ABS(PW*SIN(BTA(I)))/1000.
          ALPH=ALPH*180./3.1416
          TETA=TETA*180./3.1416
          WRITE(3,1200)I,SIGV,SIGH,PV,PH,P,ALPH,TETA,PR(I),PT(I)
       50 CONTINUE
C
      200 FORMAT(4(8F7.3/))
     1100 FORMAT(4(/3X,8F7.3))
     1150 FORMAT(/,20X,7HRESULTS,/)
     1160 FORMAT(1HI,2X,4HSIGV,4X,4HSIGH,4X,2HPV,6X,2HPH,6X,
         11HP,7X,4HALPH,3X,4HTETA,3X,2HPR,6X,2HPT)
     1200 FORMAT(1X,I2,5F8.2,1X,F6.3,1X,F6.3,2F8.2)
C
          RETURN
          END
C
          SUBROUTINE LOAD2
C
          COMMON H(8),AH(8),AV(8),BTA(8),PR(8),PT(8)
          COMMON KODE,GMA,GMS,GMW,SK,DEPTH,HW,PW,RO,SPACNG
C
C GMA IS IN PCF. PW IN LBS.
C
          A=3.1416*RO/8.
          DO 1 I=1,8
          SI=I-1
          BTA(I)=3.1416*(.5+SI)/8.
          AH(I)=ABS(A*COS(BTA(I)))
        1 AV(I)=ABS(A*SIN(BTA(I)))
          WRITE(3,1150)
          WRITE(3,1160)
C
          DO 50 I=1,8
          SIGV=PR(I)*1000.
          SIGH=PT(I)*1000.
          PV=SIGV*AH(I)*SPACNG+PW
          PH=SIGH*AV(I)*SPACNG
          IF(I-4)9,9,20
          PV=PW-SIGV*AH(I)*SPACNG
          IF(PV)11,11,12
       11 PV=ABS(PV)
          BTA(I)=-BTA(I)
       12 GO TO 20
       20 CONTINUE
          P=SQRT(PV**2.+PH**2.)
          ALPH=ATAN2(PH,PV)
          TETA=ALPH+BTA(I)
          PR(I)=ABS(P*COS(TETA))/1000.
          PT(I)=-ABS(PW*SIN(BTA(I)))/1000.
          ALPH=ALPH*180./3.1416
          TETA=TETA*180./3.1416
          WRITE(3,1200)I,SIGV,SIGH,PV,PH,P,ALPH,TETA,PR(I),PT(I)
       50 CONTINUE
C
      200 FORMAT(4(8F7.3/))
     1150 FORMAT(/,20X,7HRESULTS,/)
     1160 FORMAT(1HI,2X,4HSIGV,4X,4HSIGH,4X,2HPV,6X,2HPH,6X,
```

```
         11HP,7X,4HALPH,3X,4HTETA,3X,2HPR,6X,2HPT)
  1200 FORMAT(1X,I2,5F8.2,1X,F6.3,1X,F6.3,2F8.2)
C
       RETURN
       END
ID OF FILE
```