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PRIMARY LINING DESIGN AND SETTLEMENT ANALYSIS - EDMONTON RAPID TRANSIT SYSTEM TUNNELS

by

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FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certifies that he has read, and recommends to the Faculty of Graduate Studies and Research for acceptance, a report entitled "Primary Lining Design and Settlement Analysis - Edmonton Rapid Transit System Tunnels" submitted by W.M. Kellestine in partial fulfillment of the requirements for the degree of Master of Engineering.

Supervisor

Date: 17/6/76

ABSTRACT

This report presents the results of a design analysis for the primary tunnel lining of the proposed twenty foot diameter, twin tunnels of the Edmonton Rapid Transit System. The analysis and design focus on the spacing of the supporting ribs and the adequacy of the rib and lagging system to be used.

Several finite element analyses involving various assumptions were carried out to assess the rib and lagging system. The design of the primary lining indicates a rib spacing of four feet will be adequate to ensure the long-term stability of the lining. Some minor failure of the lagging can be anticipated if the maximum jacking pressures are required to advance the shield.

Estimates of settlement profiles above the tunnel sections are based on recorded experience with tunnels in similar soil conditions. Settlements above the tunnel are strongly dependent on the tunelling technique and workmanship involved. If good workmanship prevails, minimum settlements within tolerable limits are anticipated.

ACKNOWLEDGEMENTS

The author wishes to extend personal thanks to Dr. Z.E. Eisenstein and Dr. S. Thomson. Dr. Eisenstein suggested the topic for this study and gave valuable assistance regarding approaches to the analysis. Dr. Thomson reviewed the report, commented on the validity of the assumptions and suggested improvements which could be made. The finite element analyses discussed in this report were done by Mr. L. Medeiros, a graduate student in the Department of Civil Engineering. The author is grateful for the explanations by and discussions with Mr. Medeiros.

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INTRODUCTION

Since 1955 the City of Edmonton has been developing an extensive system of tunnels to provide new and adequate storm and sanitary sewer facilities.

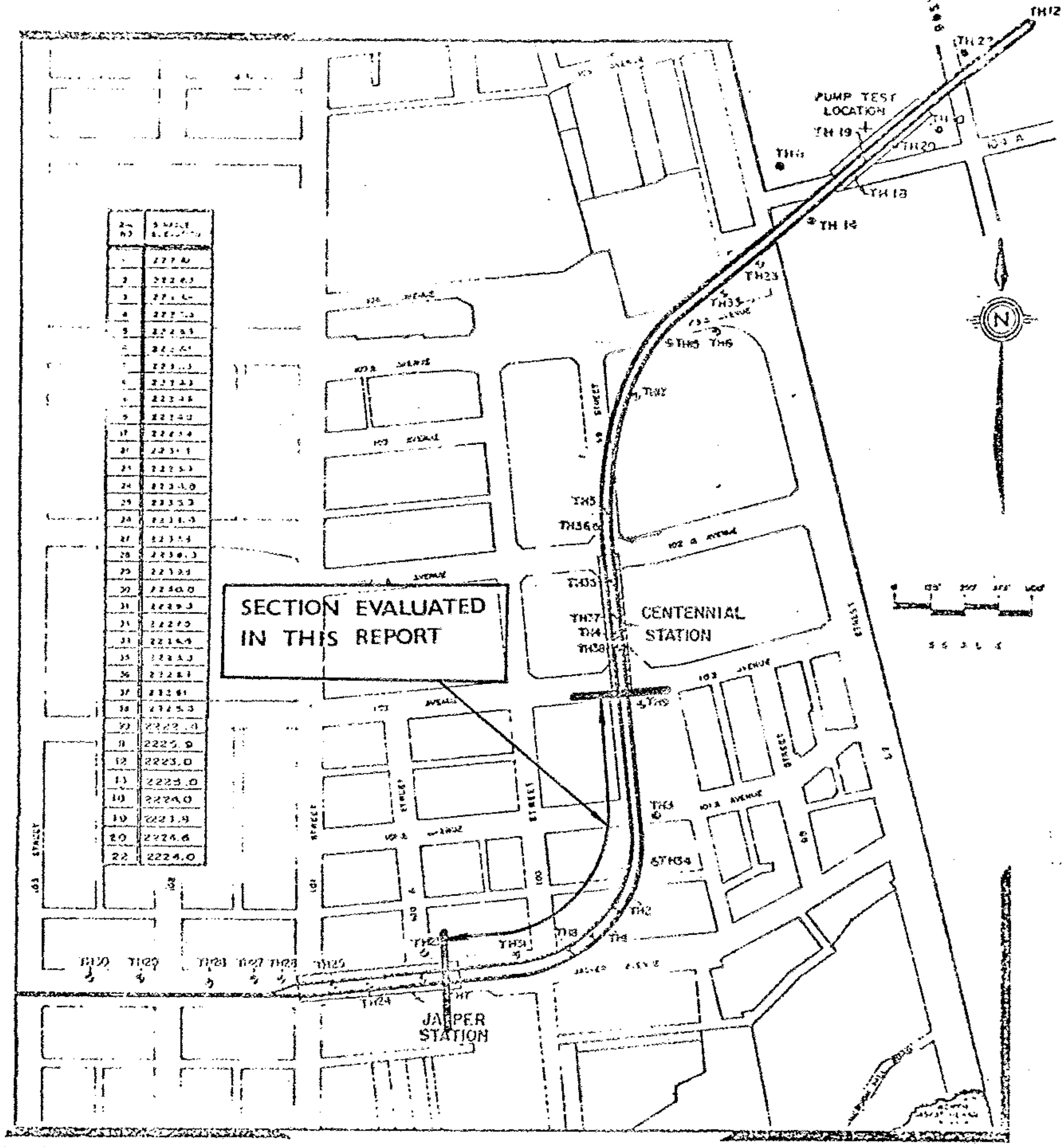
One of the main reasons for the preponderance of tunnels which have been developed in the City, is the generally conducive soil conditions which exist within the city limits and surrounding area.

The most recent major proposal for tunnelling in the City is the planned construction of a rapid transit system to service the city. Cut-and-cover techniques are proposed construction methods for a large portion of the planned route. However, in order to minimize disturbance and disruption in the central area of the city, the subway system will be constructed by tunnelling techniques in the downtown area.

This report deals with the subsurface phase of the project (Figure 1) - a portion of the North East Transit line - extending between the Jasper Avenue Station at 100A Street and 101 Avenue and Centennial Station at 102 Avenue and 99 Street.

This project includes the construction of twin twenty foot diameter tunnels at thirty-nine feet centre-to-centre. The horizontal centrelines of the tunnels vary between thirty-four and forty feet below the existing ground surface.

This portion of the rapid transit system has some specific design constraints which must be taken into account due to the route location beneath the Edmonton Plaza Hotel and several other major structures in the area.



NOTE: ALL ELEVATIONS RELATIVE TO CITY DATUM IN FEET.

PROPOSED NORTHEAST EDMONTON RAPID TRANSIT SYSTEM

(AFTER E.B.A., 1975)

FIGURE 1

SITE PLAN AND BOREHOLE LOCATION

This report does not deal with the special influences of pile/tunnel or footing/tunnel interactions but considers only the design of the primary lining under general conditions. In this respect, the final design of the primary lining obtained in this evaluation can be extended to future portions of the proposed subway system provided similar subsurface conditions exist and the construction technique is similar.

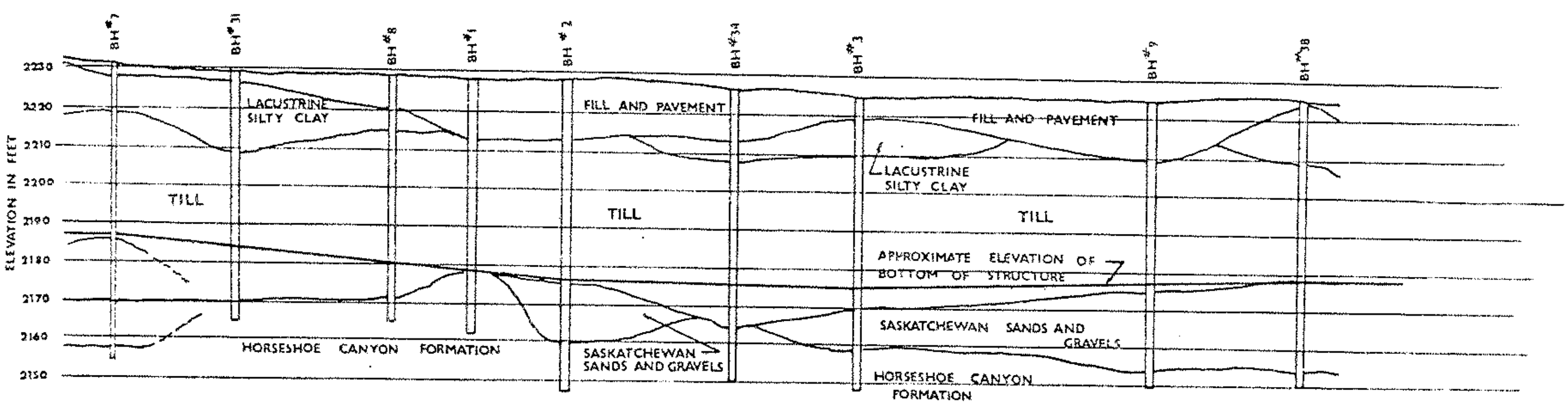
The tunnels are to be constructed using a shielded mole. The lining will be erected in two stages. First, the primary lining will be constructed as the tunnel is advanced. In an attempt to minimize the settlements above the tunnel, the primary lining will be assembled in the tailpiece of the shield and will come into contact with the soil immediately after the soil emerges from the protection of the tailpiece.

The permanent lining, consisting of cast-in-place, reinforced concrete will be poured in direct contact with the primary rib and lagging lining at a later date.

STRATIGRAPHY AND SUBSURFACE CONDITIONS

Unless otherwise noted, all information reported in this section was extracted from the geotechnical evaluation of the site by E.B.A. Engineering Consultants Ltd. (1975).

The site location plan is shown in Figure 1. A general cross-section along the proposed subway route is shown in Figure 2.



JASPER STATION TO CENTENNIAL STATION

NOTE: ALL ELEVATIONS RELATIVE TO CITY OF EDMONTON DATUM

NOTE: STRATIGRAPHY INFERRED BASED ON AVAILABLE DATA. DEVIATIONS ARE TO BE EXPECTED

HORZ. SCALE 1" = 600'

(AFTER E.B.A.,1975)

FIGURE 2

GENERAL STRATIGRAPHY

The subsurface conditions generally consist of varying thicknesses of fill overlying a lacustrine silty clay. Beneath the lacustrine deposit (or fill in some areas) is an extensive deposit of silty clay till. The fill consists of three units-upper and lower till sheets separated by an outwash sand. The upper till sheet is columnar jointed while the lower sheet has a rectangular joint system. The sand seam is up to two feet thick and is termed Tofield sand (Westgate, 1969). Its location is near the springline of the tunnel. Geotechnically the tills are essentially identical. In the vicinity of the tunnel, the till is underlain by the Pleistocene Saskatchewan Sands and Gravels which in turn overlie the overconsolidated bedrock of Upper Cretaceous Age called the Horseshoe Canyon Formation of the Edmonton Group. Further detailed information on the geology and general stratigraphy in the Edmonton area is available from Westgate (1969) and Kathol and MacPherson (1975).

The proposed invert elevation of the twin tunnels is shown in Figure 2. The tunnels generally pass through the till deposit but encounter the Saskatchewan Sands and Gravels at some locations.

Fill

The entire site is overlain by fill varying from 1 to 18 feet in thickness. The fill is predominantly remoulded silty clay but ranges from silty clay to rubble. The natural water content varies from 3 to 30 percent with an average of 20 percent. Standard penetration tests in the fill gave N value ranging from 8 to 17 blows per foot.

For design purposes a saturated bulk density of 120 pounds per cubic foot was assumed.

Lacustrine Silty Clay

The glacial lake sediments were not encountered in the vicinity of Boreholes 1, 2 and 9 but exist over the remainder of the site. The Lake Edmonton sediments are usually oxidized, greyish brown and calcerous (E.B.A., 1975). The in situ natural water content of the silty clay varies between 20 and 35 percent with 29 percent being a representative value. A saturated bulk density of 120 pounds per cubic foot was assumed for design purposes.

Till

The local till is a very dense, heterogeneous deposit varying from sandy to silty clay and containing a few boulders up to ten inches in diameter. The average natural water content of the till is 15 percent and varies from 7 to 30 percent. The average liquid and plastic limits of the till matrix are 36 and 17 percent respectively. N values measured in the till ranged from 12 to greater than 100 blows per foot with an average value of 57 blows per foot.

The undrained shear strength measured in unconfined compression tests ranged between 1500 and 9300 pounds per square foot (p.s.f.), whereas the undrained strength measured in unconsolidated, undrained tests was significantly higher - between 2500 and 21000 p.s.f.

A value of 3500 p.s.f. is recommended for design purposes (E.B.A., 1975). Recorded values of the effective strength parameter ϕ' vary; between 23° and 31.5° (Mathis, 1974) and 37.5° (E.B.A., 1975). A value of 31° was used in design.

Due to the jointing in the till (Westgate, 1969; Matheson, 1970), a cohesion intercept, C', of zero was assumed. A bulk density of 135 pounds per cubic foot is suggested for design purposes (E.B.A., 1975). The overconsolidation ratio of the till is approximately two.

One of the major difficulties anticipated during tunnel construction through the till is the possibility of running or ravelling ground due to occasional water-bearing sand pockets located within the till. The location of these sand lenses cannot be predicted.

Saskatchewan Sands and Gravels

This granular deposit is of early Pleistocene age and consists of well sorted, rounded quartz sands with minor silt and clay fractions (Kathol and MacPherson, 1975). The natural water content varies from 7 to 22 percent and the N values,as measured by the standard penetration test,are generally greater than 100 blows per foot.

Horseshoe Canyon Formation

This very dense, overconsolidated bedrock formation consists primarily of interbedded bentonitic mudstones, siltstones and sandstones. Occasional coal seams extend through the deposit. The planned subway route encounters the formation only in the vicinty of Borehole 1. In general, however, the bedrock formation will not impose any special considerations on the design of the twin tunnels in this phase of the construction.

Groundwater Conditions

The groundwater level, measured in a piezometer installed in Borehole 3, is approximately 57 feet below the ground surface, or seven feet below the proposed tunnel invert (E.B.A., 1975). Due to the impervious nature of the till through which the tunnels will be constructed groundwater conditions should not impose significant construction problems. If occasional sand and gravel pockets are encountered some dewatering may be necessary. However, the extent of these water-bearing lenses is unknown and difficult to predict.

DESIGN OF PRIMARY LINING

As mentioned previously, the lining of the twin tunnels will be composed of both a primary and a secondary lining. The primary lining must be capable of withstanding the applied loads and deformations to obviate effects of ground movements in the downtown areas.

In an attempt to avert the loss of ground due to localized failures in the till, such as that described by Matheson (1970), or through excessive tunnel squeeze, a shielded mole with an extended tail-piece will be used for the excavation of the tunnels. The primary lining consists of steel ribs and wood lagging. The ribs and lagging will be erected within the tailpiece and expanded against the tunnel as the shield is advanced (Mayo et al, 1968).

The primary lining must be designed to support both short-and long-term loadings. In the short-term, the primary lining is subjected to the action of construction loads such as axial forces caused by shield jacking. Other construction loads such as erection loadings are not usually critical to design (Mayo et al,1968).

In the long-term, the primary lining must withstand the long-term earth pressures that are transmitted to the lining.

The design of the lagging has been separated into the following four categories;

A - Rib Supports	(i)	Short-Term
	(ii)	Long-Term
B - Lagging	(i)	Short-Term
	(ii)	Long-Term

A literature review found that specific cases of primary lining design are not well documented. One of the reasons for this is that one of the least quantifiable inputs influencing the design of the linings is the workmanship involved.

One of the more extensive overall outlines of factors important to tunnel design and construction was reported by Peck (1969).

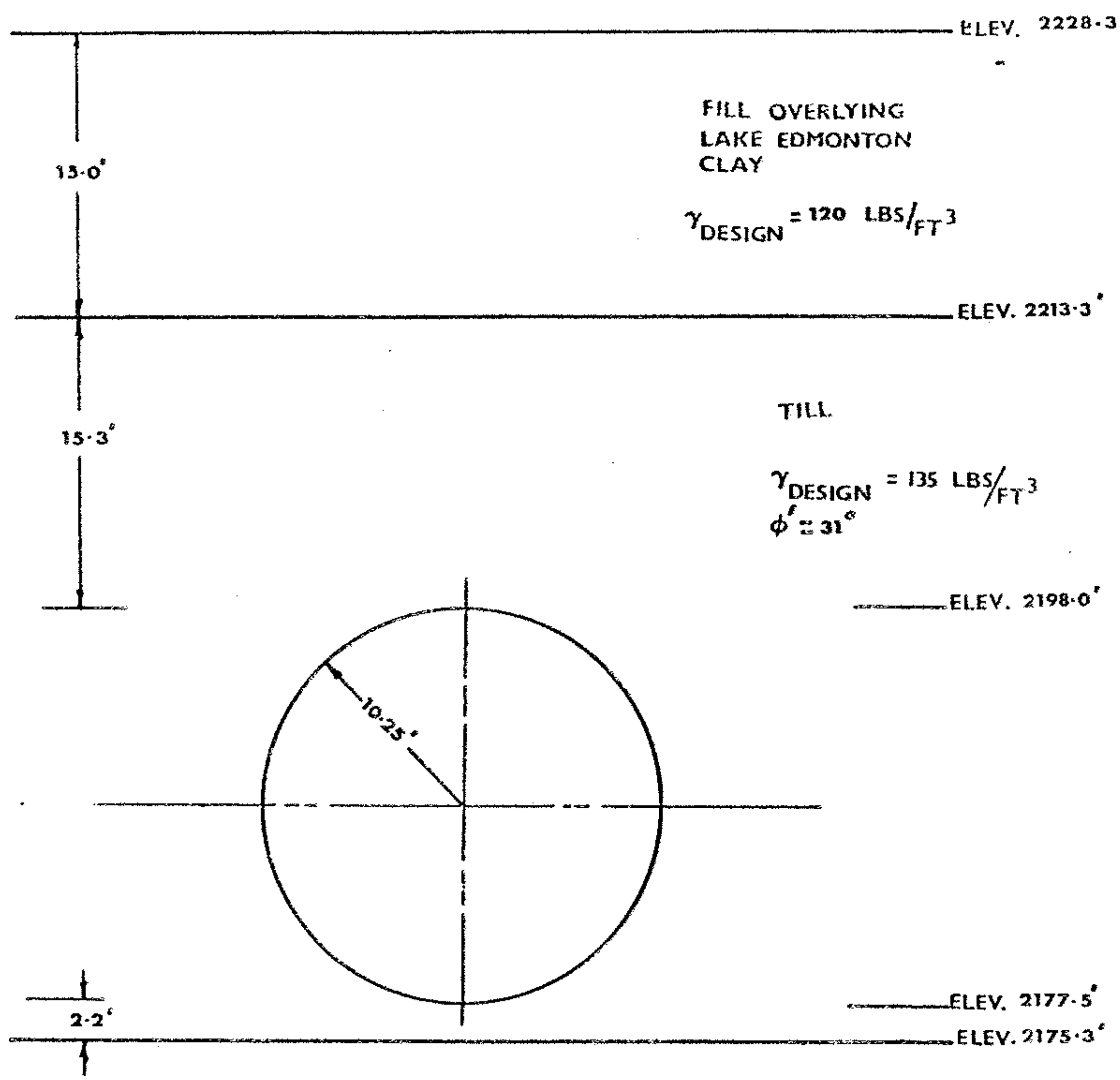
Several methods of estimating the earth pressure acting on a tunnel lining are available. If it is assumed that the lining is completely flexible and can deform to a neutral position, then the distribution of pressure on the lining is as shown in Figure 4. In this case the capacity of the lining must be adequate to carry the ring

SUBSURFACE CONDITIONS AT BOREHOLE 2

130' N. OF JASPER AVE., 200' E. OF 100 STREET

U.T.M. 5,934,211·0 N

33,917·8 E

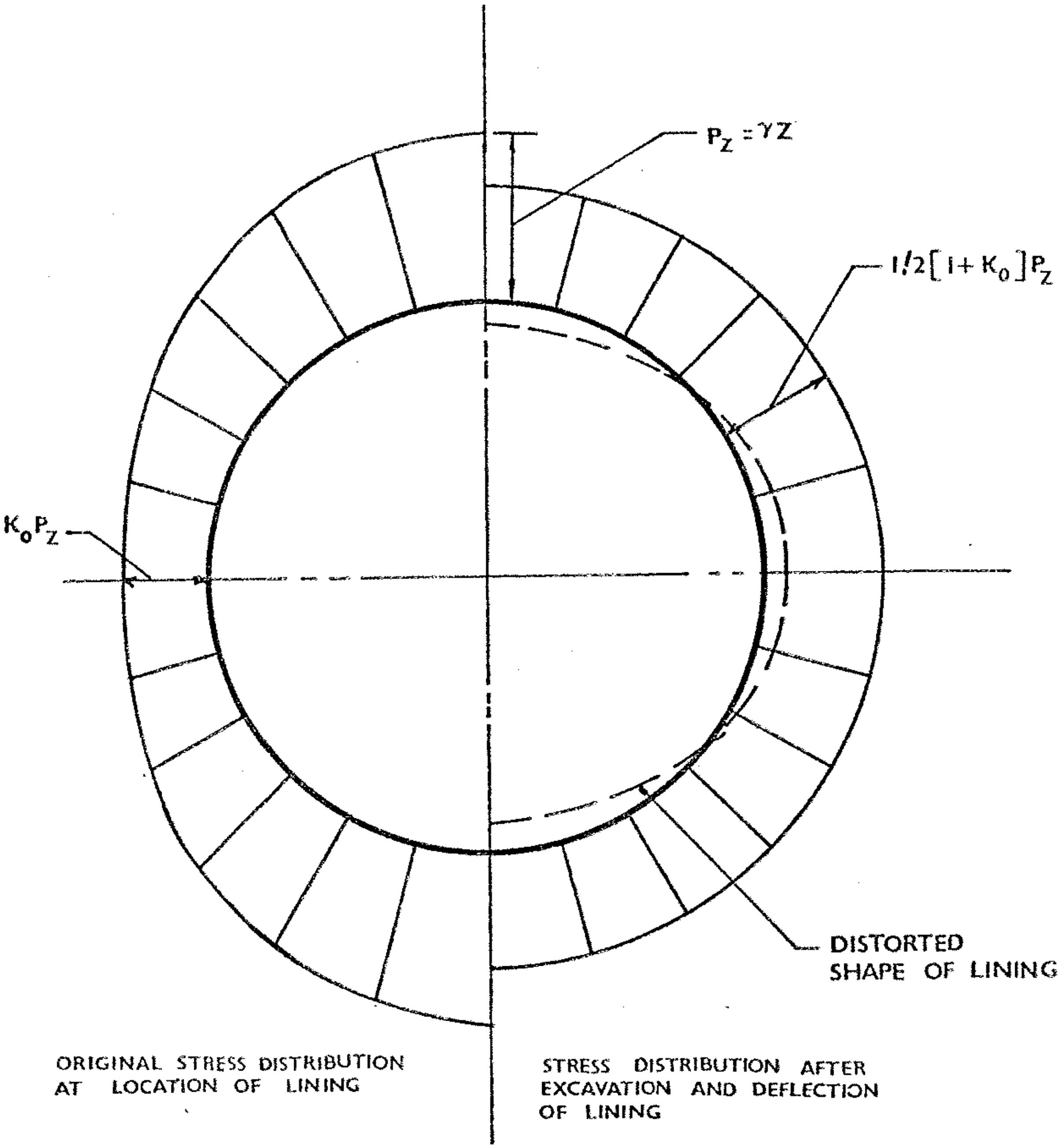


SCALE 1" = 10'

(AFTER E.B.A., 1975)

FIGURE 3

ASSUMED SUBSURFACE CONDITIONS FOR PRIMARY LINING DESIGN



(AFTER PECK, 1969)

FIGURE 4

PRESSURES AGAINST AND DEFLECTIONS OF HYPOTHETICAL LINING stresses from the applied radial pressure. However, if the member is flexible, it will deform such that no bending moments exist within the structure.

Rib Design: Long-Term

The standard rib supports used in constructing tunnels of this size in the City of Edmonton are WF6x25 beams with a yield stress of 50,000 p.s.i.

The assumed soil stratigraphy used in the analysis of the primary lining is shown in Figure 3.

As outlined by Peck (1969), the flexibility of a rib and lagging, primary tunnel lining lies somewhere between complete flexibility and perfect rigidity. Since the rib supports are installed in four segments and the efficiency of the connections is low, an assumption of near perfect flexibility is valid. The lining therefore can deform to a position where a uniform stress distribution exists, that is, bending moments are zero.

Using this premise of a flexible lining under the influence of the pressure distribution shown in Figure 4, the design of the rib spacing is worked out in detail in Appendix A - Design Notes. The basic steps involved are (i) an assumption of the stress distribution acting on the lining and (ii) determination of the physical properties of the ribs to be used in the construction. From these data the proper rib spacing is calculated.

Peck (1969) suggests that the stresses acting on the primary lining after deflection can be estimated using the relationship

$$\sigma = \frac{1}{2} (1 + K_o) p_z \qquad (1)$$

where p_z is the total overburden pressure at the elevation of the centreline of the tunnel.

In order to use this relationship an estimate of K_o must be made. Brooker and Ireland (1965) suggested that K_o could be estimated for a soil if the stress history and plasticity index were known. An average I_p of 20% and an O.C.R. of two yields a value of K_o approximately equal to 0.70 (Brooker and Ireland, 1965). The relationship

$$K_o = 0.7 + 0.1(\text{O.C.R.} - 1.2) \qquad (2)$$

was suggested by Bowles (1974).

Using the above equation K_o is equal to 0.78. Other studies of the behaviour of the till in the Edmonton area (Matheson, 1970) suggest a K_o of unity. For design purposes a value of $K_o = 1$ was used. This may tend to give conservative results, however, it probably represents the upper limits of K_o in the downtown Edmonton area. In addition, the requirement for minimizing the ground movements suggests a conservative approach.

Using a yield strength of 50,000 p.s.i. for the steel and the properties of the wide flange section (C.I.S.C., 1970; see also Appendix A), the allowable axial load that the rib can resist is 220 kips.

Using the stress distribution from equation (1) and an allowable axial load of 220 kips per rib, a spacing of 4.1 feet centre-to-centre is obtained. (Appendix A). As pointed out in the Design Notes (Appendix A), using the assumptions noted, no moments or shear exist within the member.

The Factor of Safety involved in the design is applied through the Factor of Safety in the design of the steel rib. The recommended allowable yield stress is 0.6 Fy or 30,000 p.s.i., This corresponds to a Factor of Safety of 1.67 in the design.

Terzaghi (1943) proposed an alternative method of estimating the applied load exerted on the tunnel lining. Széchy (1967) outlined several other methods of calculating the pressure distribution on a tunnel lining. These methods deviate from the previous assumptions and calculations in that they assume the full overburden pressure does not develop due to arching in the soil. When the tunnel is excavated and movement takes place in the surrounding soil, arching develops over the tunnel and the actual pressures on the tunnel supports are less than the in situ pressures (Yardley, 1970).

Terzaghi (1943) developed the following relationships for arching over tunnels in cohesive soils.

$$\sigma_v = B_1 \left(\frac{\gamma - \frac{C}{B_1}}{K \tan\phi} \right) \left(1 - \varepsilon^{- K \tan\phi \, D/B_1}\right) \; .... \qquad (3)$$

$$\text{where } B_1 = [B_o + H \tan(45° - \tfrac{\phi}{2})] \text{ ....} \tag{4}$$

and γ = unit weight of soil

B_o = half width of the tunnel opening

D = height of overburden above tunnel

H = height of the tunnel (diameter in this case)

K = coefficient of lateral earth pressure

ϕ = angle of shearing resistance

C = cohesion intercept

Substituting the appropriate geometrical and physical properties in this relationship, the vertical stress acting on the top of the tunnel, σ_v, is approximately 20γ. This is roughly one-half of the value used in the design of this report. The methods outlined by Széchy (1967) lead to results similar to the above.

Peck's (1969) assumptions were used for the design of the rib spacing since the form is convenient and conservative results are obtained.

The rib spacing of 4.1 feet, as calculated previously, represents the long-term condition after deformations have taken place.

Rib Design: Short-Term

In the short term design of the ribs substantially different loading conditions exist. An asymmetrical loading condition exists while the primary lining is being formed in the tailpiece of the shield. Further, due to the jacking forces required to advance the shield the load may vary with time.

The analysis of the rib is given in detail in Appendix A. However, the assumed rigidity of the stiffening rib is critical to the final outcome. Under conditions of assumed perfect fixity a failure situation may exist where the stresses in the web exceed the allowable stresses by a factor of 3. However, since the lagging is not rigidly connected to the WF beam and the beam is not fully constrained against rotation (a pin or hinge connection is assumed) development of stresses in the web due to moments will be minimal.

Design of the rib support under these conditions is in fact a complex three dimensional problem. Simple statics have been applied to this loading situation and the results are in error but probably on the conservative side.

Lagging Design

The lagging must be designed to withstand the applied loads and undergo only reasonable deformations in both the short-and long-term cases.

The lagging that will be used in this phase of the tunnel construction is 4" x 6" spruce lagging (Personal Communication G. Emanuel, 1976). The lagging will be placed between the flanges of the ribs such that the 4" width of the lagging is in the 6" spacing of the flanges.

The strength properties assumed for the lagging are dependent on the type of load application. All properties and formulae used in the analysis were obtained from the Timber Design Manual (1972).

Lagging Design : Long-Term

In the long-term loading case the lagging can be considered as a simply supported beam under the influence of a uniformly distributed load.

Assuming earth pressures equivalent to those used for design of the rib spacing the required lagging section modulus is 3 1/2 times that which is proposed for use. This is in direct conflict with the current practice and performance of tunnelling operations in Edmonton, hence a re-evaluation is presented in the following paragraphs.

The relative stiffness of the primary lining components can be expressed in the following manner.

$$\frac{E_{timber}}{E_{steel}} \simeq \frac{1}{25}$$

Therefore, when considering the rib and lagging system a significant contrast in the relative stiffness of the members exists.

Re-evaluating the assumptions for design of the rib and lagging system, reconsider a single cell in the length of the tunnel consisting of rib/lagging/rib. Due to the relative stiffness of the members some arching of the soil between the ribs develops as the lagging deforms.

Again, using Terzaghi's theory of arching in cohesive soils (1943) and accounting for the continuous nature of the tunnel lining ($B_1=B_0$) substitution in equation (3) gives:

$$\sigma_v = 1.66\ B_1\ \gamma \quad \text{where } B_1 = 2.05 \text{ ft.}$$

Further, Terzaghi and Peck (1967) suggest that the zone of influence of arching does not exceed a height greater than 2.5 to 3 times the width of the opening.

In order to design against the worst anticipated condition a rectangular stress distribution of $\sigma_v = 2.5\ B\gamma$ (where B = 4.1 ft) was assumed to act on the lagging. Detailed calculations of the long-term design of the lagging are given in Appendix A. Under the long-term symmetrical loading conditions the lagging has a Factor of Safety of 1.05 in bending and 1.14 in shear. Further, correction factors have been applied to reduce the allowable strength in the long-term (see Design Notes, Appendix A).

In the long-term condition, considering the lagging as a simply supported beam, 4" x 6" spruce lagging is adequate in terms of both bending and shear.

Lagging Design: Short-Term

The short-term design of the lagging is more critical however.

Using the proposed construction technique, the mole is advanced by propulsion jacks which jack against the last stiffening rib which has been placed. This induces considerable axial load in the member and further enhances the maximum fibre stresses set up due to the bending of the lagging.

The maximum jacking load which can be developed for advancement of the mole is approximately 3500 tons. The anticipated working range of jacking loads varies from 500 to 1000 tons (Personal Communication, G. Emanuel, 1976).

For timber members under the influence of both axial loads and bending moments the following criterion must be met for design (L.T.I.C., 1972).

$$\frac{M/S}{F'_b} + \frac{P/A_N}{F'_a} \leq 1 \ldots \quad (5)$$

where
M = bending moment, inch-pounds
P = axial load, pounds
S = section modulus, inches^3
A_N = net area, inches^2
F'_b = allowable working stress in bending, psi
F'_a = allowable direct working stress, psi

Considering the applied jacking loads as uniformly distributed pressures around the ring, the following results are obtained:

LOAD	$\frac{M/S}{F'_b} + \frac{P/A_N}{F'_a}$
3500 tons	$4.70 \nleq 1$
1000 tons	$1.52 \nleq 1$
500 tons	$0.99 \leq 1$

Therefore, if the jacking force required to advance the mole exceeds 500 tons, failure of the lagging might be anticipated. However, the recommended allowable stresses for the spruce lagging used in the design do, on a statistical basis, contain a Factor of Safety of approximately two (Personal Communication, Dr. J.G. Longworth, 1976).

It would be within reason therefore to anticipate that in the normal operating range of 500 to 1000 tons the performance of the lagging will be acceptable.

Moreover, the analysis detailed in Appendix A assumes that deformation and arching develop immediately. In reality, in the short-term, the uniformily distributed load acting on the lagging will be somewhat less than the design value

However, if it is necessary to develop the full design jacking load of 3500 tons to advance the mole, then some failure of the lagging will occur. This failure of the lagging should occur within the first one or two cells adjacent to the mole. The farther the lagging is from the shield the less axial load impinged upon it since some of the load is dissipated as shear stress along the outer perimeter of the tunnel at the interface between the soil and lagging.

SETTLEMENT

During the construction of the subsurface portion of the Edmonton Rapid Transit system (Figure 1), the amount of settlement that will take place is critical.

While all the buildings which are to be tunnelled under will have their foundations underpinned, any subsidence above the tunnels may cause unacceptable damage to adjacent structures. Moreover, settlement of the ground above the tunnels may cause lack of support and cracking may develop in the floor slabs of the buildings which have been underpinned. Utility and service connections are also susceptible to damage due to settlement.

The construction of any tunnel represents a change in the state of stress in the soil mass accompanied by subsequent displacements. The settlements associated with tunnel excavation can be broadly broken into two categories. Those associated with the inevitable displacements caused by the change in the state of stress and those caused by the construction techniques or workmanship.

The strains and displacements are necessary and unavoidable in tunnel construction. Without allowing deformation to take place, the arching effect in the soil, which reduces the applied loads on the tunnel to a reasonable value would not exist.

In order to minimize the amount of subsidence above the tunnel, the primary lining will be assembled within the tailpiece and "extruded" as the shield is advanced. The lining will be expanded into contact with the soil and key blocks used to make the final joint connection. One disadvantage of the system being used in Edmonton is that the lagging width is 4 inches and the spacing between the flanges on the WF beams is approximately 6 inches. Thus, it is necessary to block or wedge the lagging into contact with the soil. This operation will have a significant influence on the amount of subsidence taking place above the tunnels.

The nature and amount of settlement occurring above a tunnel is strongly dependent upon the type of ground through which the tunnel is passing and the groundwater conditions during construction. If good workmanship prevails, it has been found by observation of various tunnels that the settlements are usually symmetrical about the vertical centre-line of the tunnel.

The shape of the settlement curve above a tunnel is a trough-like depression. This depression can be roughly approximated using the error function or probability curve (Peck, 1969). The validity of this approximation has been confirmed through model tests on laboratory scale tunnels (Atkinson et al, 1974).

However, in order to define the properties of the normal distribution curve it is necessary that the characteristics of the distribution be known. To fully define the settlement trough a measure of the maximum anticipated settlement above the centreline of the tunnel is required along with a measure of the standard deviation of the curve.

Figure 5a gives the properties of the settlement trough above a single tunnel in terms of δ_{max} and i (equivalent to standard deviation).

Peck (1969) has assembled from observations of tunnels through various soil types, a chart which can be used to estimate the standard deviation of the settlement trough distribution. From Figure 5b, knowing the depth and radius of the tunnel, an estimate of i can be made for a specific soil type. The curve representing soft to stiff clays was used for the analyses in this report. In reality, the properties of the till will lie between the classification for rock and hard clays and the soft to stiff clay group. Using the soft to stiff clay classification should yield over-estimates of the subsidence above the tunnel.

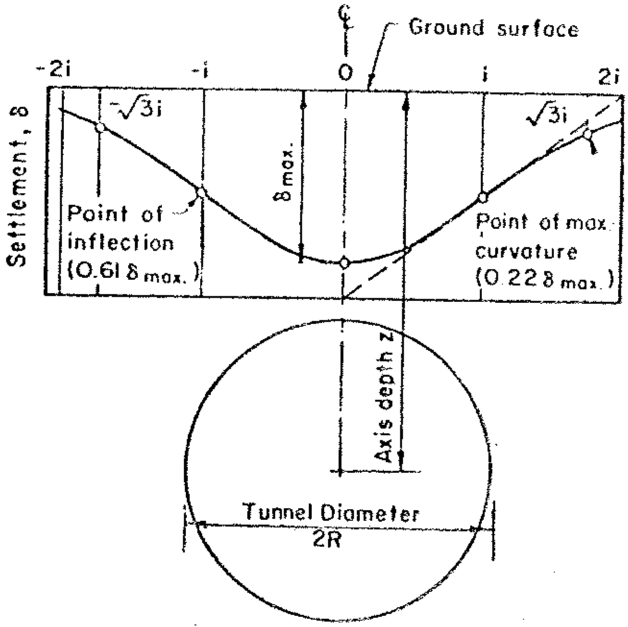
Since the value of the maximum settlement at the centreline of the trough, δ_{max}, is not known, settlement profiles have been determined for various percentages of the theoretical volume of the tunnel excavation.

Using the following relationship, the maximum settlement, δ_{max}, can be calculated for any assumed volume.

$$V \simeq 2.5 \ i \ \delta_{max}. \quad \ldots \qquad (6)$$

Single Tunnel

Calculations, based on the guidelines established by Peck (1969) for settlements above a single tunnel, are outlined in Appendix A.



Ratio $\frac{i}{R}$ is function of $\frac{z}{2R}$ and soil conditions
Volume of trough $\simeq 2.5\, i\, \delta_{max.}$

FIG. 5A Properties of Error Function or Normal Probability Curve as Used to Represent Cross-Section Through Settlement Trough Above Tunnel

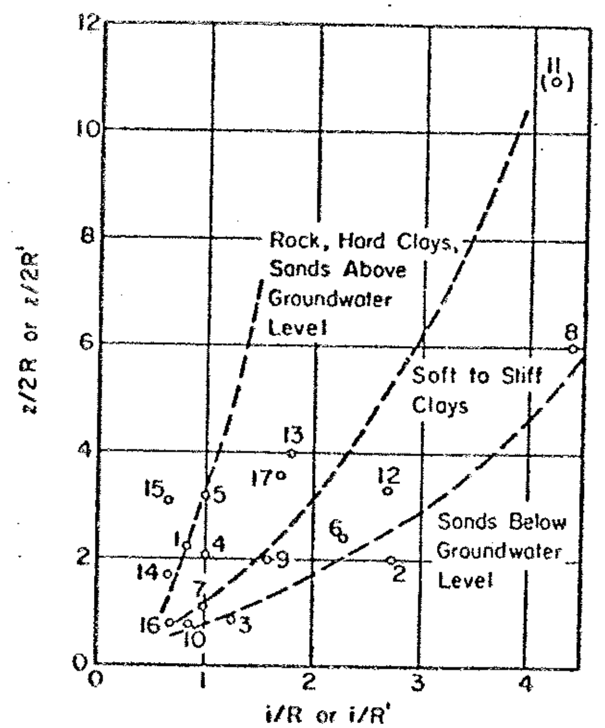


FIG. 5B Relation Between Width of Settlement Trough, as Represented by i/R, and Dimensionless Depth of Tunnel, z/2R, for Various Tunnels in Different Materials

(AFTER PECK, 1969)

FIGURE 5

PROPERTIES OF SETTLEMENT TROUGH

The estimated settlement profiles for a single tunnel are plotted in Figure 6.

The maximum percentage of lost ground assumed is equivalent to 15 percent of the tunnel volume. This amount of subsidence is excessive and unexpected on this project. The results were plotted to indicate the influence on settlement of large runs into the tunnel should they occur.

It can be seen that using the relationships described by Peck (1969), the magnitude of the percent lost ground has little influence on the width of the settlement trough at the ground surface. This is so since i, a measure of the standard deviation of the distribution, is only influenced by the soil type and not the amount of lost ground.

From Figure 6, the width of the settlement trough over which significant displacements may take place is approximately 80 feet for a single tunnel.

Settlement Above Twin Tunnels

There are two methods available for determining the settlement profile above a pair of twin tunnels:

(i) Superimpose the settlement above two tunnels.

or (ii) Calculate a distribution based on using an equivalent radius, R', calculated from the following

$R' = R + \frac{d}{2}$ where d = spacing of the twin tunnels, centre-to-centre.

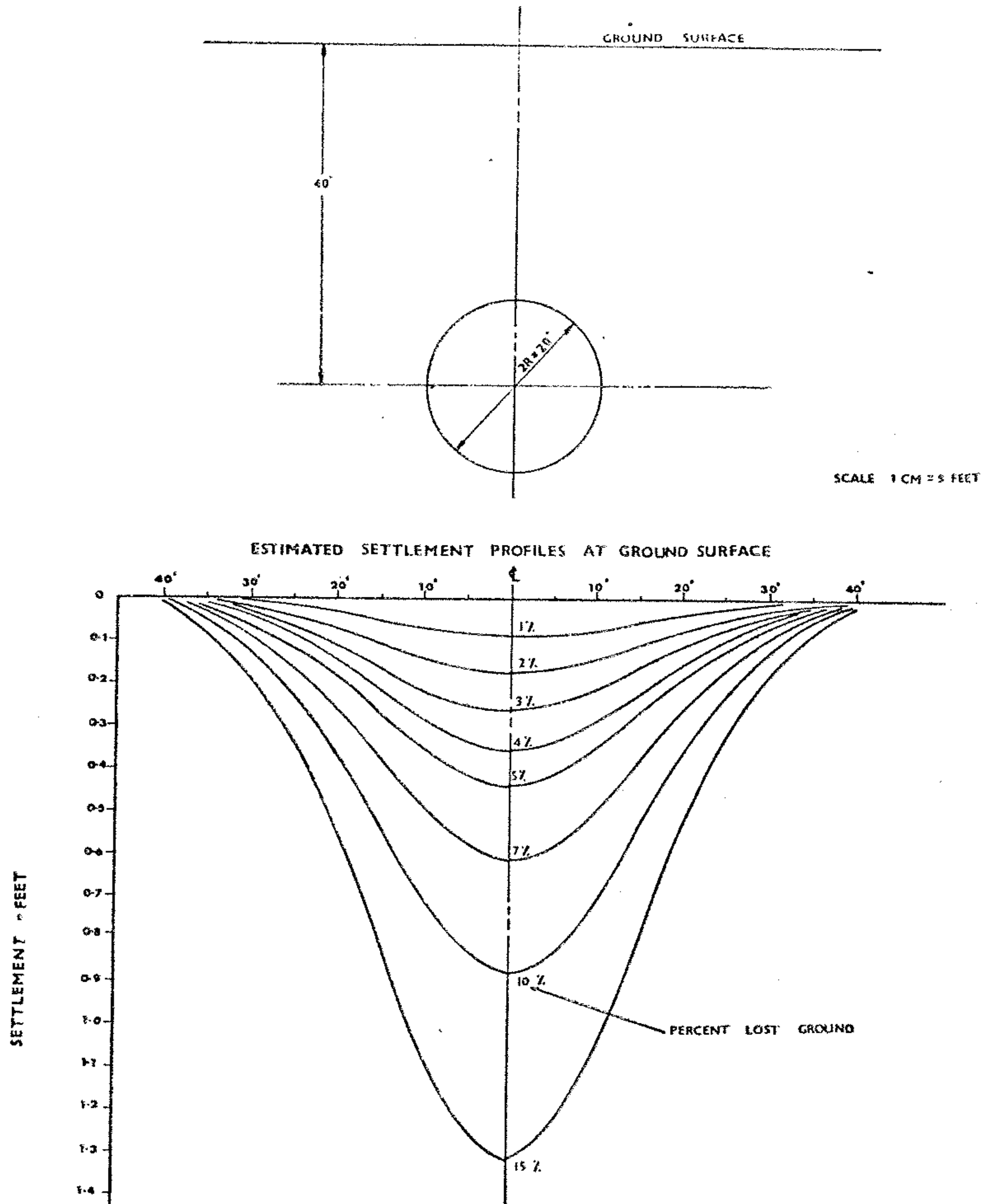


FIGURE 6

ESTIMATED SETTLEMENT PROFILES
SINGLE TUNNEL

The curves resulting from superposition of single settlement curves are plotted in Figure 7.

On the basis of an equivalent radius R', equal to 30 feet, the settlement profiles are shown in Figure 8. All calculations are summarized in Appendix A.

It can be seen that in both cases the width of the settlement trough extends approximately 60 feet on either side of the centreline of the twin tunnels. The width of the trough and the amount of settlement that takes place above the trough have severe implications for the structures adjacent to Jasper Avenue and 99 Street. Extensive loss of ground into the tunnels can cause settlements of from 0.1 to 0.2 feet at distances of 50 to 60 feet from the centreline. If this amount of subsidence took place, substantial damage to adjacent structures would occur.

The matter of a single or double settlement troughs is not easily defined. Depending on the soil conditions and tunnel spacing either could result. Since the twin tunnels are passing through a stiff till material it is anticipated that the settlement curves obtained from the superposition technique will be valid.

The amount of settlement is also very much a function of the ground condition; if ravelling ground or soft clay were encountered during the excavation, larger settlements would be anticipated. The shielded mole to be used in the construction of the tunnels allows only partial exposure of the face under adverse conditions. Thus, severe running or ravelling of ground into the excavations can be inhibited

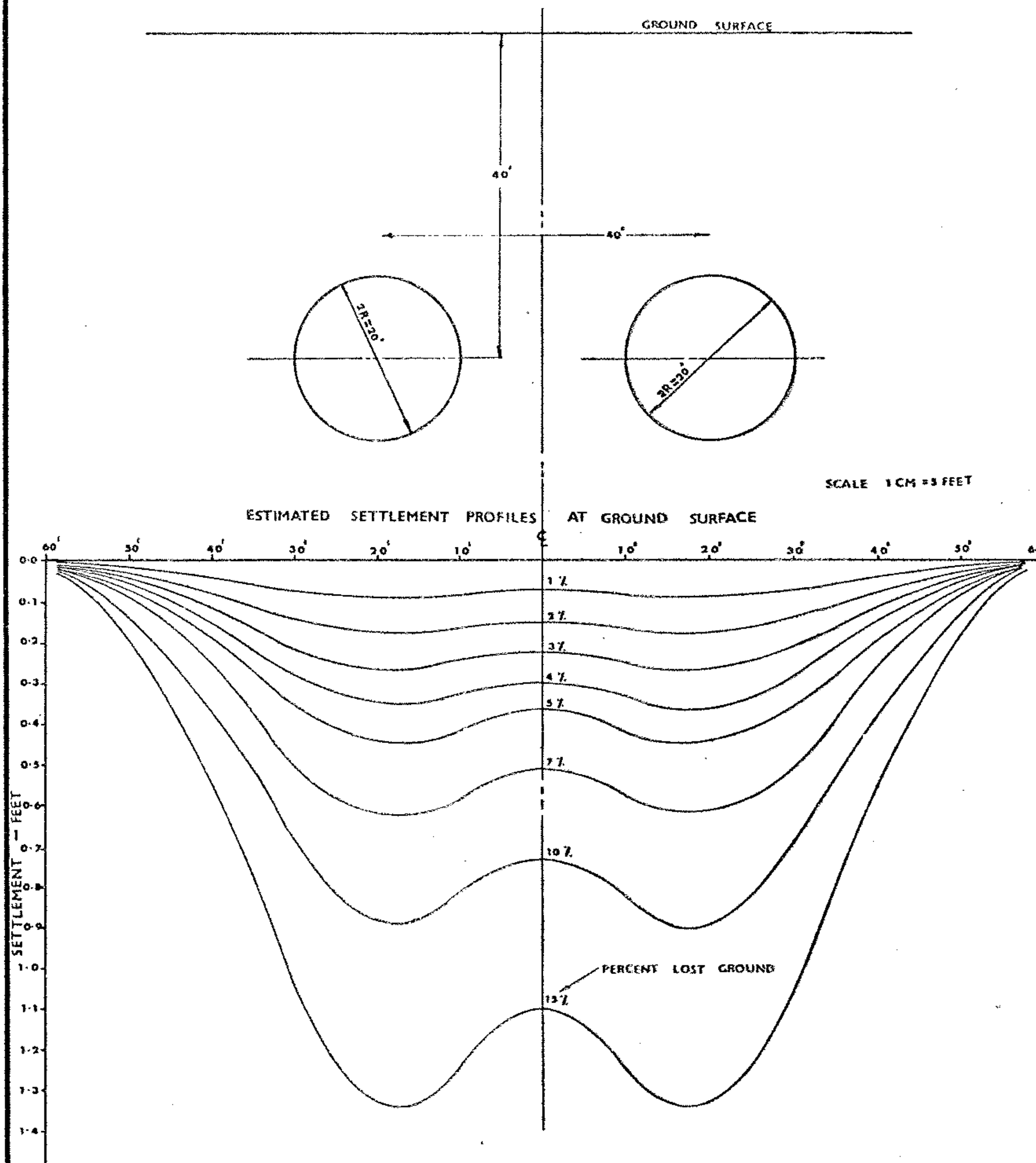


FIGURE 7

ESTIMATED SETTLEMENT PROFILES ABOVE TWIN TUNNELS USING SUPERPOSITION

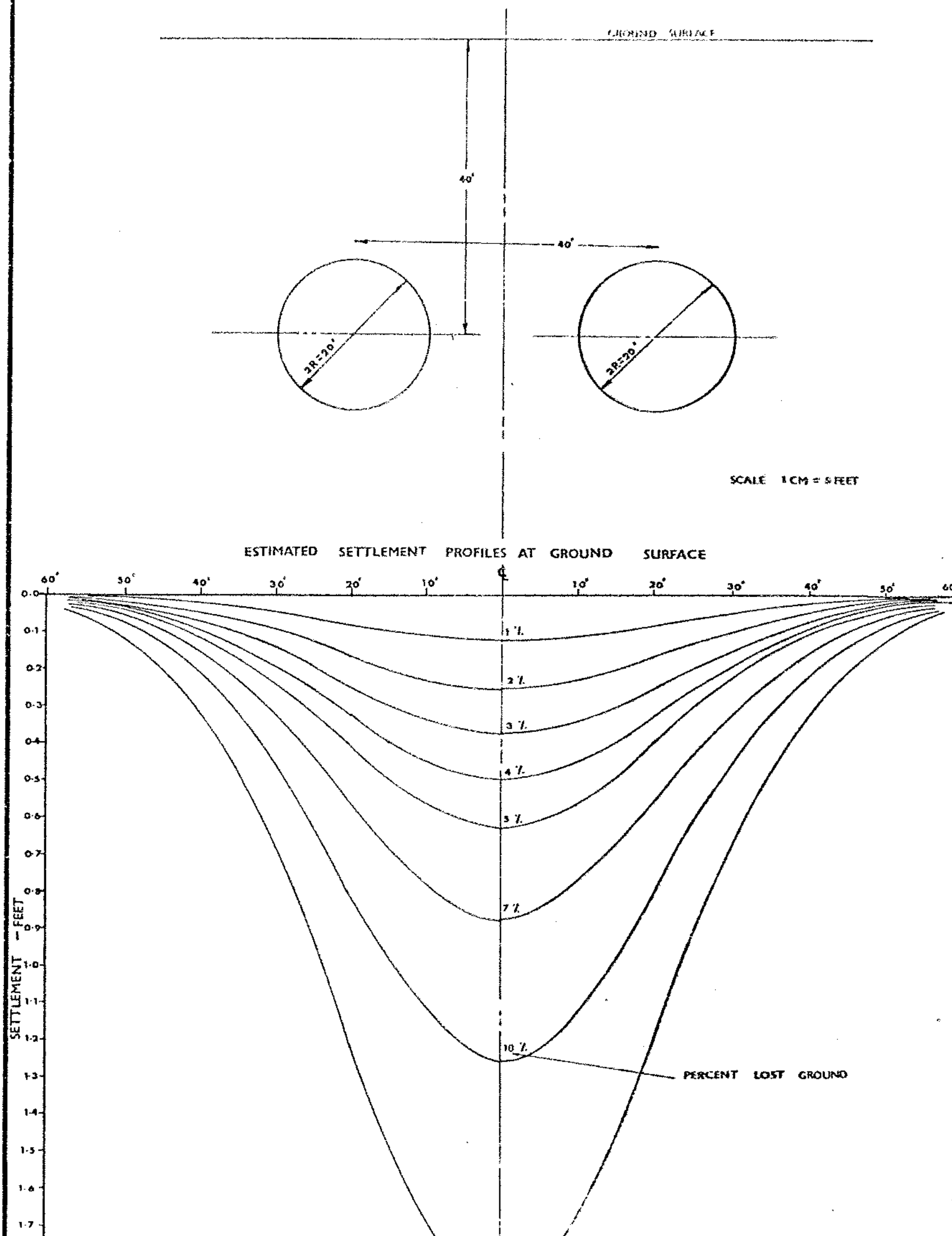


FIGURE 8

ESTIMATED SETTLEMENT PROFILES ABOVE TWIN TUNNELS USING EQUIVALENT RADIUS APPROACH and the associated settlements minimized.

Larger diameter tunnels in Edmonton usually have associated with them small settlements at the surface (Personal Communication, G. Emanuel, 1976). However, most experience has been gained with tunnels located at greater depths in the Horseshoe Canyon Formation.

In an attempt to estimate the amount of settlement which may take place the construction techniques and soil conditions must be fully evaluated. Further, the problem is complicated by a variable stratigraphy across the site. While the tunnels pass entirely through the till, the varying thicknesses of the lacustrine silty clay and fill across the site will give variable settlements along the tunnel section.

As mentioned previously, the construction technique involves a primary lining which comes into direct contact with the soil as soon as the soil is exposed behind the tailpiece. If improper wedging of the lagging takes place a maximum squeeze or closure of the soil of approximately two inches into the tunnel may occur. This amounts to 1.5 percent of the theoretical volume of the tunnel. From Figure 7, the induced settlements will amount to approximately 0.11 feet at the centreline of the twin tunnels, 0.1 feet 30 feet from the centreline and about 0.03 feet at distances of 50 feet from the tunnel centrelines.

The effect of the passing shield in the second tunnel may also influence the deformations (and settlements) above the first tunnel. However, since the till is a relatively stiff material, (Cu approximately 3500 psf) remoulding of the soil should be confined to the region immediately bordering the tunnel and the influence on the adjacent tunnel will be negligible.

From the case histories outlined by Peck (1969) for tunnelling in similar ground conditions, where good workmanship prevails, maximum settlements of from 0.1 to 0.2 feet were recorded. This corresponds to a settlement trough volume of from 1 to 2 percent of the excavated volume.

Finite Element Analysis

In an earlier design evaluation, in an attempt to evaluate the stress changes and deformations taking place around the tunnel, several finite element analyses were carried out. Both stress analyses and deformation analyses were performed.

The simplified soil profile, with the appropriate elastic properties which were used, is shown in Figure 9. The finite element grid used in the immediate vicinity of the tunnels is shown in Figure 10. The entire grid used actually represented a section 450 feet long by 100 feet deep. However, results of the analyses showed that the zone of influence of the tunnel excavation was restricted to the immediate vicinity of the tunnels.

Stress Analysis

The stress analysis was conducted in order to estimate the deformations which would occur in the soil due to the excavation of the tunnel. In these analyses, two different assumptions were made

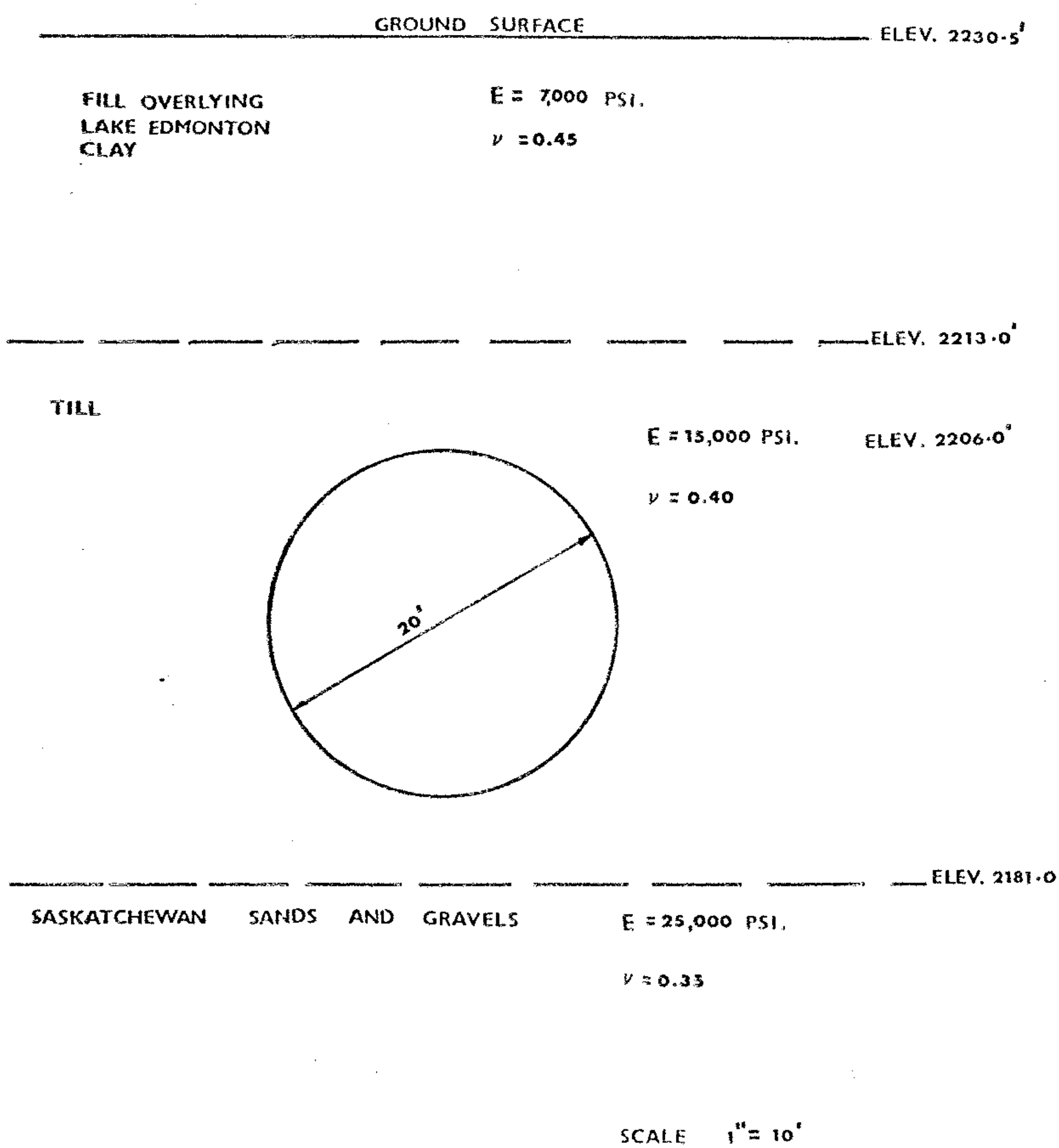
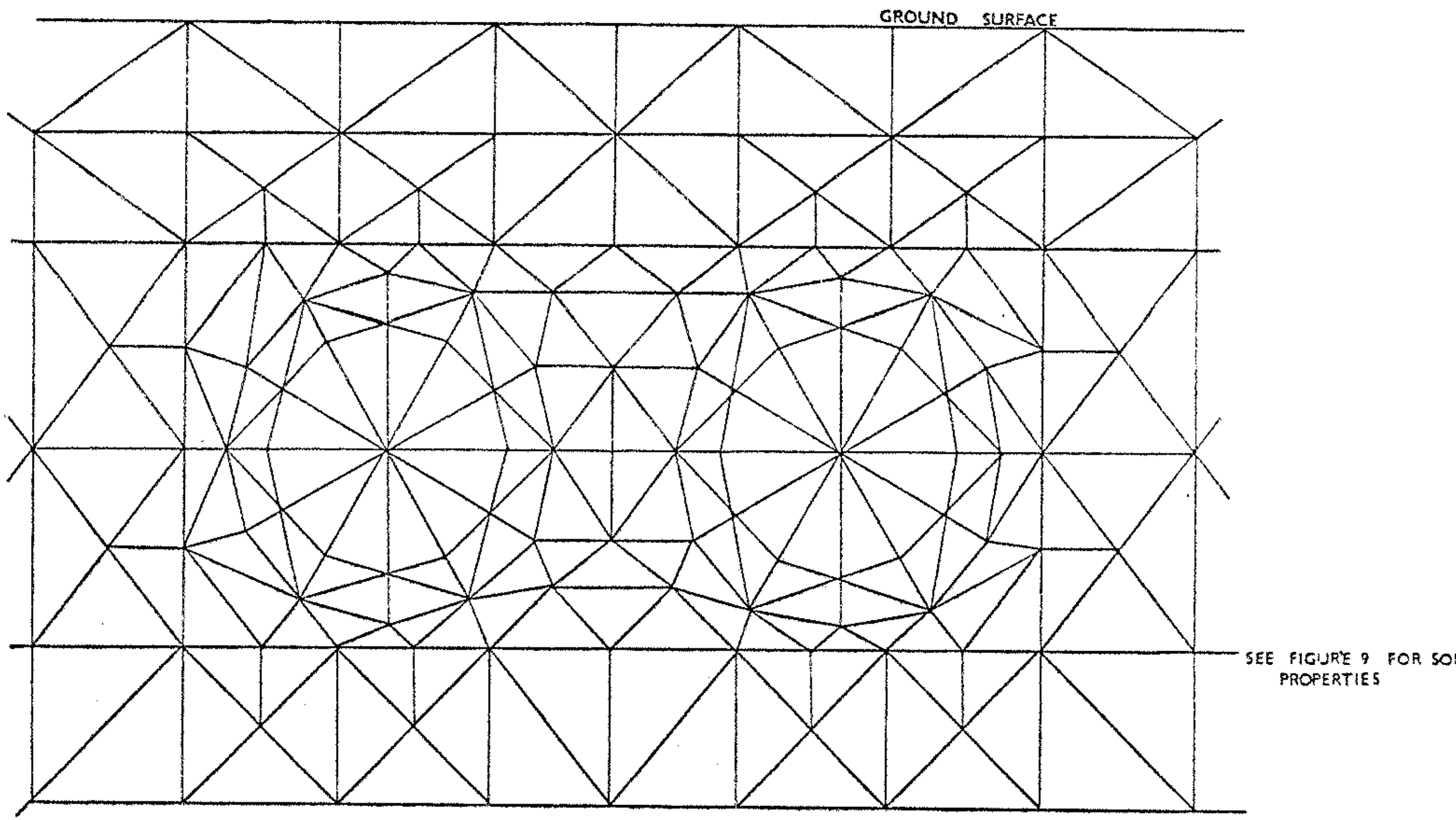


FIGURE 9

SIMPLIFIED SOIL PROFILE FOR FINITE ELEMENT ANALYSIS



SCALE 1" = 10'

FIGURE 10

FINITE ELEMENT ANALYSIS GRID IN IMMEDIATE VICINITY OF TUNNELS regarding the stiffness of the lining. Analyses were carried out using a soft (flexible) lining where

$$\frac{E_{lining}}{E_{till}} \simeq \frac{1}{36}$$

and a stiff lining where

$$\frac{E_{lining}}{E_{till}} \simeq 4$$

This was an attempt to define the influence of Peck's (1969) assumption of a flexible, primary lining on the deformations occurring above the tunnel. The forces applied to the nodes are tabulated in Appendix A and correspond to the stress distribution suggested by Peck (1969) and shown in Figure 4.

The results of these two analyses are plotted in Figures 11 and 12. Using an elastic analysis, the calculated deformations above the tunnel are negligible - in the order of 10^{-8} feet.

The analyses do confirm the likelihood of a symmetrical settlement profile represented by the normal distribution curve. At a depth of 8.5 feet below the ground surface no displacements were calculated in the analyses.

In the analyses carried out, an error was made in the direction of application of one of the nodal forces. This error was such that it would tend to over-estimate the displacements above the tunnel. Further, the zone of influence of the individual forces is

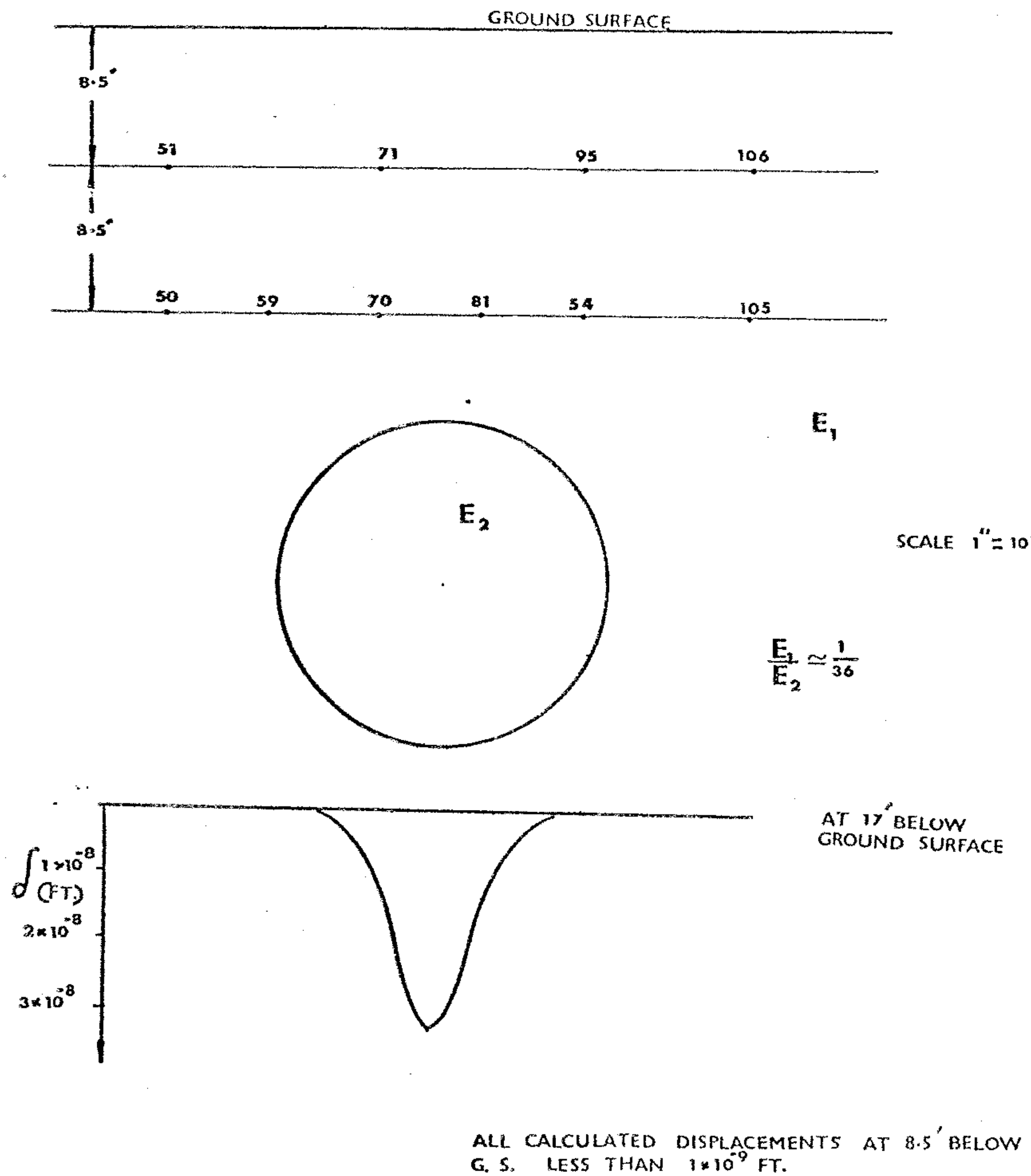


FIGURE II

VERTICAL DISPLACEMENTS ABOVE TUNNEL
STIFF LINING ASSUMED IN TUNNEL

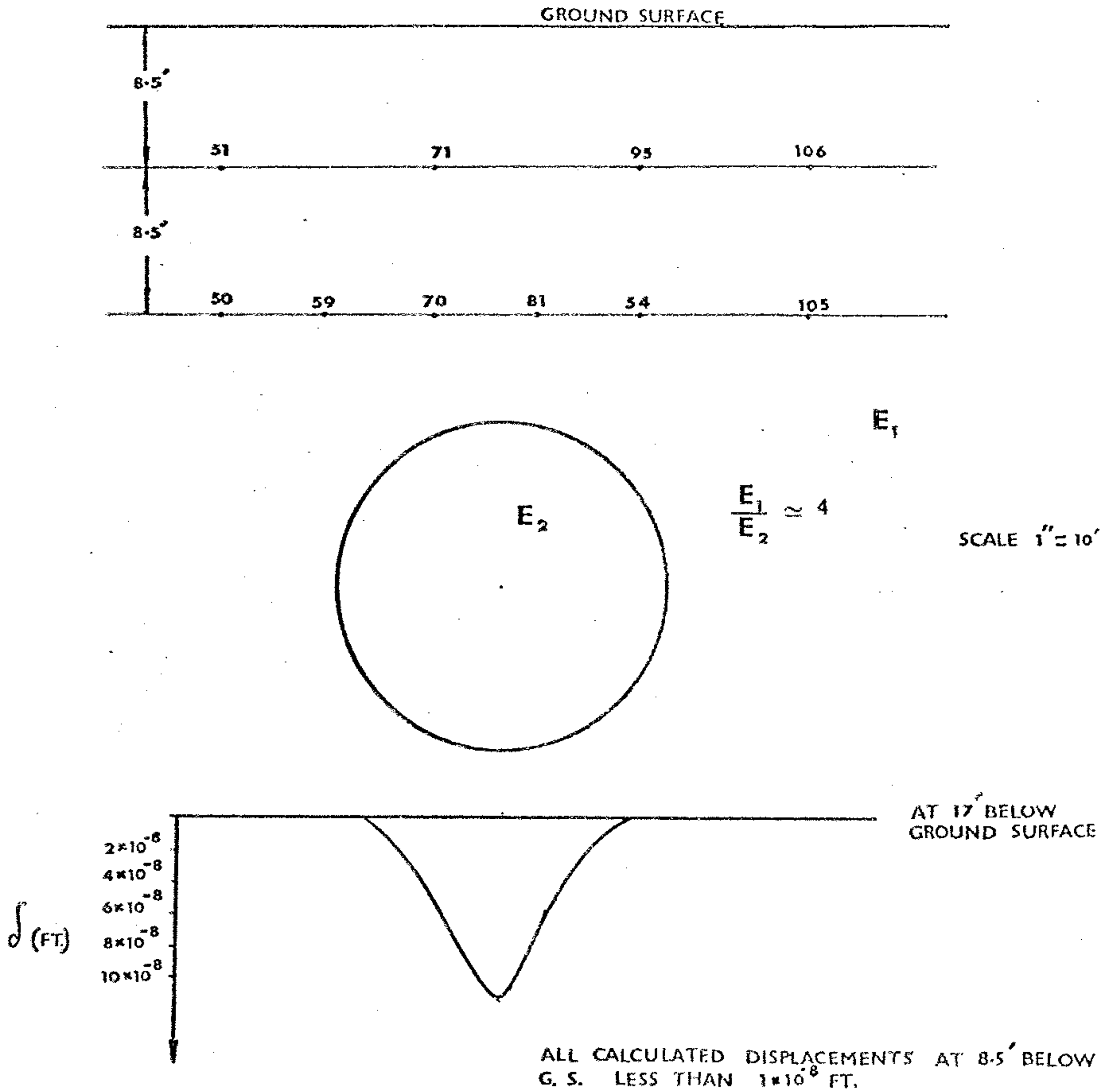


FIGURE 12

VERTICAL DISPLACEMENTS ABOVE TUNNEL
FLEXIBLE LINING ASSUMED IN TUNNEL

relatively confined and the implications of the error are therefore not serious.

Deformation Analysis

This analysis was carried out on the premise of a two inch radial squeeze into the tunnel after the shield had passed. This was felt to be the worst case but does not represent any lost ground due to running or ravelling at the face.

Displacements of two inches radially inwards were applied to the nodes. The x-y components of these displacements are calculated in Appendix A. The results of this analysis did not prove conclusive to any degree. The effect of the two inch radial displacement when used with a linear elastic model of the soil was to develop a localized area of high stresses immediately adjacent to the tunnel. No influence was felt at the ground surface.

In an attempt to reconcile the zone of high stresses, after the first analysis a zone of plasticity was delineated by studying the stress levels and the available shear strength. An average displacement of 1.1 inches was found at the nodes bordering this zone. Since the zone of plasticity was above the available strength in the soil, then the displacements of 1.1 inches were input acting on a circle in the mesh where the size of the circle represented the zone of plasticity.

Using this approach, at a depth of 8.5 feet below the ground surface, displacements were calculated and are shown in Figure 13. Again, no deformations were calculated at the ground surface.

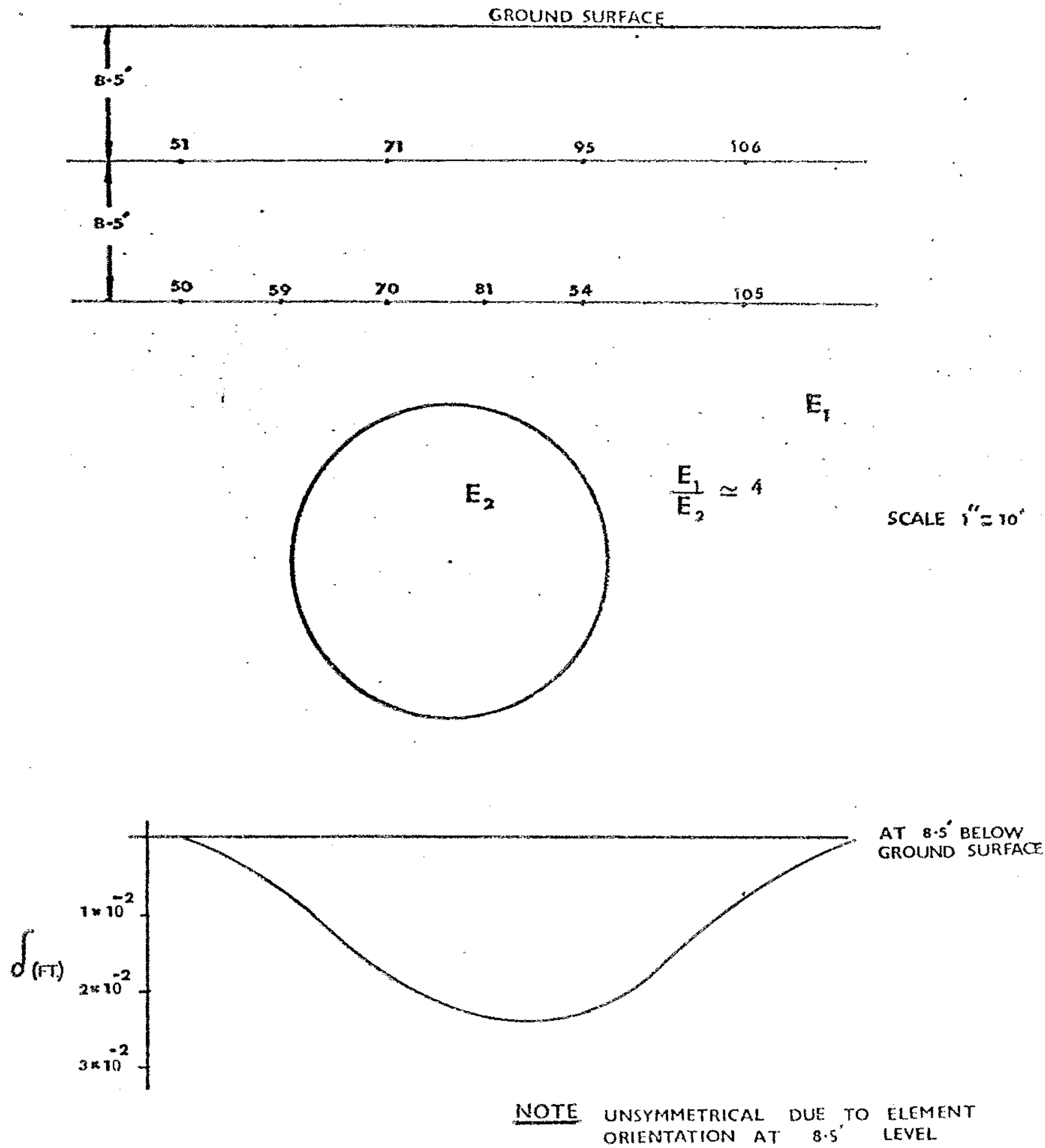


FIGURE 13

VERTICAL DISPLACEMENTS ABOVE TUNNEL
1·1" DISPLACEMENT AROUND ZONE OF PLASTICITY

Discussion of Finite Element Analysis

Finite element analyses, applied in the method described previously do not give representative or realistic results. The major problem is that the analysis is based on a linear elastic model for the soil. Using this approach, the soil, when under the influences of large stress changes or large deformations, acts as though infinite strength is available.Because of the non-linear properties of the soil stress-strain relationships, the use of a linear model for the tunnel analysis is not valid particularly in the case of the displacement analyses.

As an example of the stress levels in some elements, the maximum shear stress developed in one element during the displacement analysis was in excess of 38000 p.s.f. The drained shear strength of the soil in the vicinity of this element, at a depth of approximately 40 feet, is 4000 p.s.f. The shear stress induced by the displacement of two inches is approximately 10 times greater than the available shear strength.

In order to obtain representative results from a finite element analysis of the tunnel, a non-linear stress-strain model of the soil must be used. Further, an incremental analysis would assist in obtaining reasonable results.

Due to the limited resources available these necessary refinements to the analysis were not carried out.

Anticipated Settlements

The actual deformations taking place above the tunnels will probably lie between the empirical approach of Peck (1969) and the results of a reasonable finite element analysis. Due to the consequences of any settlements, the empirical approach of Peck (1969) should be used as a guideline in design even though it probably represents an over-estimate if good workmanship and favourable ground conditions prevail.

SUMMARY AND CONCLUSIONS

Analysis of the steel stiffening ribs indicates that a spacing of 4 feet will be adequate. Various methods are available for estimating the earth pressures which will act on the tunnel. The worst possible cases have been used in design in order to obtain conservative results. The pressures actually existing on the tunnel will probably be substantially less than those used in the analysis.

The 4 x 6 inch spruce lagging is capable of supporting the anticipated loads in the long-term loading condition. In the short-term however, if the maximum jacking load of 3500 tons is required to advance the mole, failure of the lagging must be anticipated.

The settlements taking place above the tunnel are estimated to be in the order of 0.5 to 1.5 percent of the tunnel volume, assuming good workmanship and no running or ravelling of ground occurs. Minor problems may be encountered with the occasional water-bearing sand seam or pocket within the till stratum.

Construction techniques including wedging of the lagging, and the rate of advance of the tunnelling operations will control the amount of deformation to a large extent. Unfortunately, these represent the least quantifiable components in the basic analyses carried out.

Finite element analyses must be extended to include non-linear stress-strain relationships and incremental analysis if reliable estimates of the deformations are to be obtained.

RECOMMENDATIONS

Construction of the Edmonton Rapid Transit system tunnels provides an excellent opportunity to obtain information to re-evaluate the design processes and assumptions at least on a local basis.

A properly conceived instrumentation program could be used to obtain valuable information on earth pressures acting on the tunnel, deformation of the lining and subsidence or settlement above the tunnel.

Since the extent of this tunnelling project is so large, measurements and performance evaluations could be used as a basis for re-design of the tunnel lining as work progressed.

The design approach in this report took the most conservative approach possible. If accurate measurements were made the design could be reduced to an efficient state and result in substantial savings in the overall cost of the tunnelling project.

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APPENDIX A

- DESIGN NOTES -

Design Notes Tunnel Project

Assume that the temporary lining is completely flexible, (and connections are not 100% efficient) thus the temporary lining will deform such that the neutral axis in the stiffening rib coincides with the line of thrust and no bending moments will develop in the rib (in a plane normal to the axis of the tunnel).

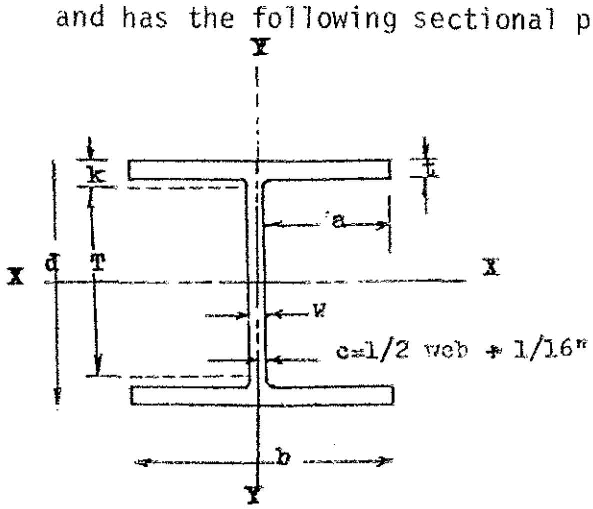
Only radial pressures acting on the rib.

Design of Spacing of WF Ribs Within the Tunnel

The standard rib to be used in construction of the primary lining for the rapid transit tunnel is:

WF6 x 25 @ 50 ksi yield

and has the following sectional properties (C.I.S.C., 1970)



Area = 7.35 in^2

dw = 2.04 in^2

h/w = 17.1

d/A_f = 2.30 in^{-1}

wt = $25^{\#}$/ ft

d = 6.37"

b = 6.08"

t = 0.456"

w = 0.320"

a = 2 7/8"

T = 4 1/2"

k = 15/16"

k_1 = 9/16"

c = 1/4"

I_x = 53.3 in^4

S_x = 16.7 in^3

r_x = 2.69 in

Z_x = 18.9 in^3

I_y = 17.1 in^4

S_y = 5.63 in^3

r_y = 1.53 in

<u>Design Notes</u> <u>Tunnel Project</u>

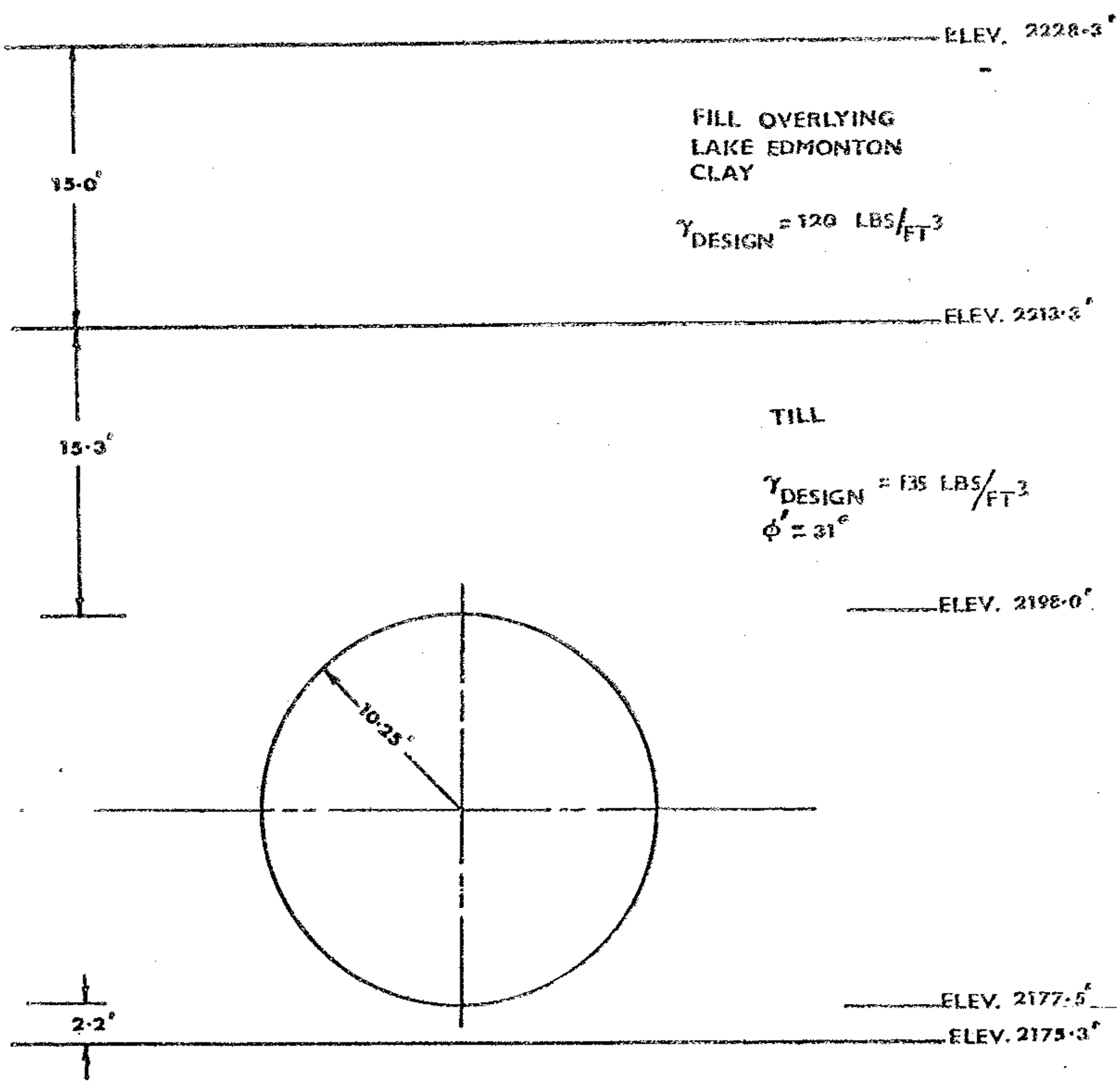
Assume the following critical geologic profile and conditions at the site:

Borehole No. 2

130'N of Jasper Avenue 200'E of 100 Street

U.T.M. 5,934,211·0 N

33,917·8 E



Design Notes   Tunnel Project

Soil profile obtained from E.B.A. Geotechnical Evaluation (E.B.A., 1975) Water table at depth but ground saturated (assumed) Tunnel diameter of 20.5' from details of proposed mole.

K_o Value

The till in downtown Edmonton area is known to be overconsolidated. However, significant lateral stress relief due to influence of downcutting of Saskatchewan River Valley reduces the coefficient of earth pressure at rest to $K_o \approx 1.0$

From Brooker and Ireland (1965)

$$K_o = f_n(I_p \text{ and O.C.R.})$$

from E.B.A. (1975) I_p avg = 20%

O.C.R. = 2

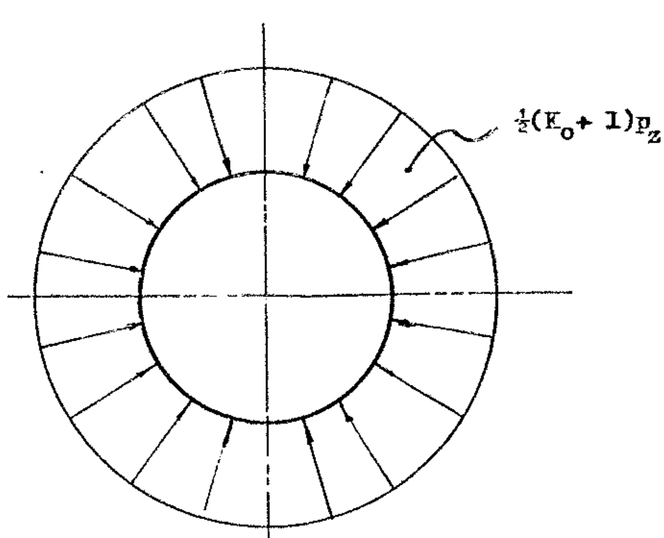
From Figure 11 (Brooker and Ireland, 1965) for I_p = 20%, O.C.R. = 2

$$K_o = 0.70$$

Therefore, if K_o = 1.0 used for design results will be conservative.

Pressure Distribution on Tunnel Lining

From Peck (1969) have the following suggested design pressure distribution around a flexible tunnel lining (see Figure 4).



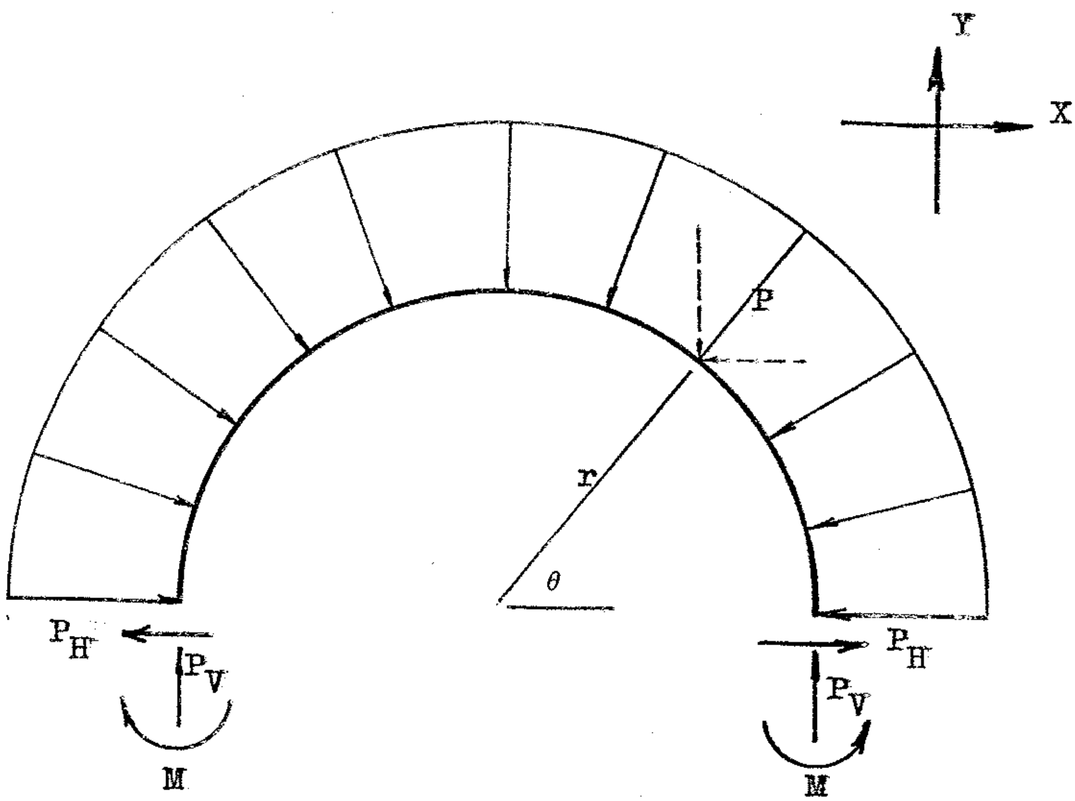
Where P_z = the vertical stress at the centre of the tunnel (used as an average value)

If $K_o=1$ then the uniform applied stress on the primary lining

$$= 1/2\ (1+1)\ P_z = P_z$$

Basis of Analysis

Consider a 1/2 section of the tunnel since it must be in equilibrium. The forces can be resolved in the following manner:



Now considering equilibrium in the horizontal direction

$$\Sigma F_x = o \qquad P_h = P_h$$

and $\qquad P_h = {}_o\int^{\pi} rp\cos\theta d\theta = rp\,{}_o\int^{\pi} \cos\theta d\theta$

$$= rp\ [-\sin\theta]_o^{\pi} = rp\ (o-o)$$

therefore we see that $P_H=P_H=o$ and equilibrium exists in the horizontal direction.

Design Notes   Tunnel Project

In the vertical direction

$$2P_v = \int_0^{\pi} rp \sin\theta \, d\theta$$

$$= rp \int_0^{\pi} \sin\theta \, d\theta$$

$$= rp[\cos\theta]_0^{\pi}$$

$$= rp[\cos\pi - \cos o] = -2\,rp$$

therefore $P_v = rp$

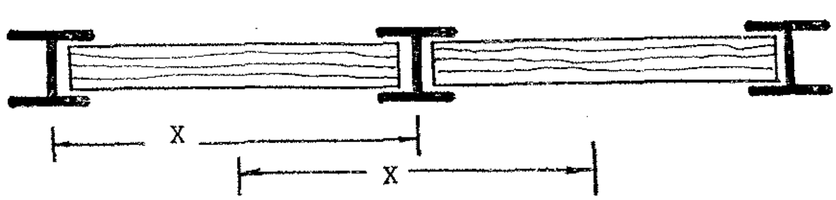
From moment equilibrium it can be shown that the moments at the ends are also zero.

Further, since a uniform all-round state of stress exists then no matter where the section is taken around the circumference of the tunnel, the shear force across the section and the moments will be zero.

Therefore, the only force acting on the section is:

$$P_v = rp \text{ where } p = \frac{1}{2}(1 + K_o)p_z$$

Now consider the system



Each stiffening rib has transferred to it via the lagging a load dependent on the spacing of the ribs.

Design Notes   Tunnel Project

The allowable stress in the steel member is:

$$\sigma_{all} = 0.6\ F_y = 0.6(50{,}000) = 30{,}000\ \text{psi} = 30\ \text{ksi}$$

Since σ_{all} = 30 ksi and the area of the section is 7.35 in^2

Therefore $P_{all} = \sigma_a \cdot A = 30{,}000 * 7.35 = 220$ kips.

From the previous calculations know $P_h = R\cdot p$ and $p = \gamma z$ (z @ ℄ of tunnel)

and in this case $p = \gamma_1 z_1 + \gamma_2 z_2 = 120(15) + 135(25.6)$

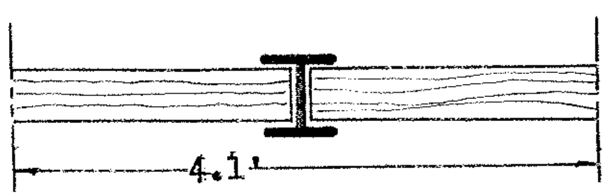
$$= 5260\ \#/ft^2\ (\#/ft/ft\ \text{of tunnel})$$

Therefore $P_h = R\cdot p = 10.25\ (5260) = 53900$ #/ft of tunnel

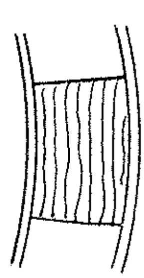
Therefore the spacing of the ribs based on the axial compressive strength of the WF beam alone is:

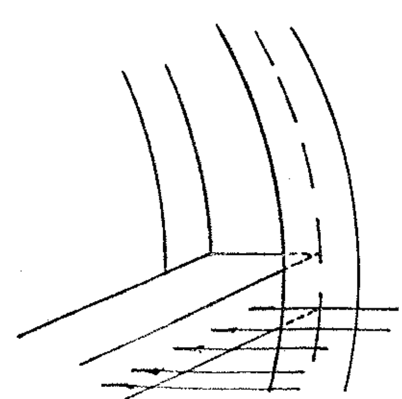
$$\text{Spacing} = \frac{P_{all}}{P_h} = \frac{220000}{53900} = 4.1\ \text{feet.}$$

Now consider the lagging/beam interaction



Considering the lagging board at the side of the tunnel

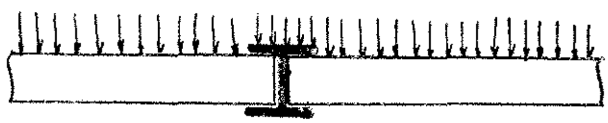




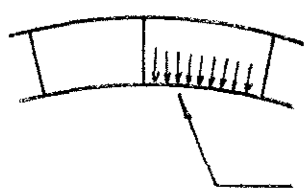
Design Notes   Tunnel Project

Say the boards are 12" pieces of lagging (N.B. this assumption makes no difference to this portion of the analysis)

Then, the following condition exists



or on the flange of the beam (assume all load transferred to lower flange)



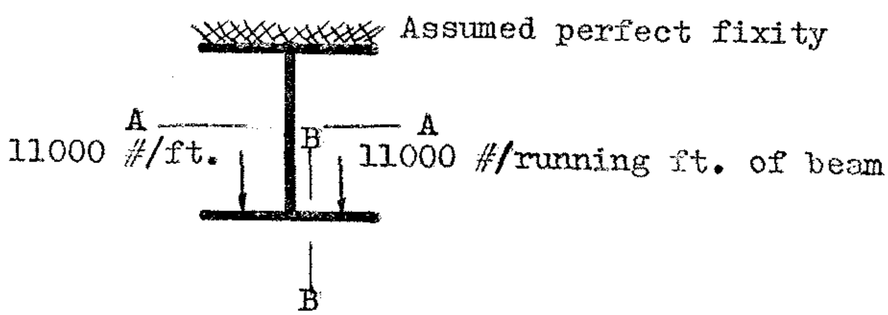
Total load Q over 1 ft of beam for a 4.1 foot rib spacing is

$Q = K_o \gamma z * \frac{L}{2}$ (for each 1/2 web)

Therefore $Q = 1.0(5260)\ (\frac{4.1}{2}) = 10783\#$ - say 11,000 $^{\#}$/ft of beam

Now if the upper flange of the beam is assumed a rigid support for the

Long-Term Condition the following condition exists:



Considering section A-A under symmetrical loading

$P = 22000\ ^{\#}/\text{ft of beam.}$

Area of web for 1 foot = $w * \ell = 0.320" * 12 = 3.84\ in^2$

Therefore $\sigma_{actual} = \frac{22000}{3.84} = 5730$ psi = 5.7 ksi

$\sigma_{all} = 0.6\ F_y = 0.6 * 50$ ksi = 30 ksi

Therefore there is adequate strength in the web to resist the applied loads.

Now considering the section B-B through the flange

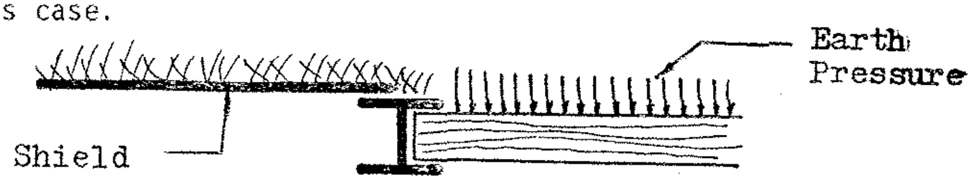
$$\tau_{actual} = \frac{11000}{Area} \quad \text{where Area} = 0.456" * 12" = 5.74 \text{ in}^2$$

$$= \frac{11000}{5.47} = 2010 \text{ psi}$$

But $\tau_{all} = 0.6\ F_y = 30000$ psi therefore flange strength O.K.

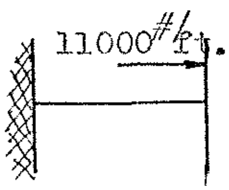
During the passing of a 2nd tunnel adjacent to an existing tunnel, it has been shown (Peck, 1969)that the tunnelling alters the shear stress distribution around the first tunnel and increases the applied load on the lining of the first tube. However, in the cases considered by Peck (1969) the applied pressures approached the total overburden pressure. Therefore, in this case since K_o=1 has been assumed for design then the design has been based on full overburden pressure. Therefore, one can neglect the effect of the passing tunnel on the design of the stiffening ribs in the primary lining.

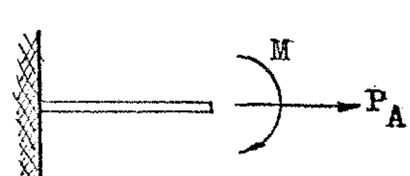
Now consider this case.



This case may arise during tunnelling, i.e. where the stiffening ribs have been left unsupported and there is only an applied load from the lagging on one side.

Considering a gross approximation





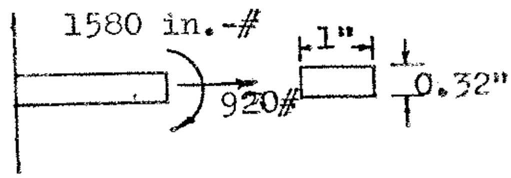
Design Notes Tunnel Project

Where $M = P*L$ where $L = \frac{(2.875 - 9/16)}{2} + 9/16 = 1.72"$

Therefore $M = \frac{11000}{12} * 1.72" = 1580$ in#/in of beam

and $P_A = 920$ #/in of beam

Therefore considering a rectangular section 1" wide



$\sigma_{max} = \frac{P}{A} + \frac{My}{I}$

$I = \frac{bh^3}{12} = \frac{1*(0.32)^3}{12} = 0.00273 \text{ in}^4$

$= \frac{920}{0.32} + \frac{1580(0.16)}{0.00273} = 95500$ psi

$\sigma_{min} = \frac{P}{A} - \frac{My}{I} = 90{,}000$ psi

but $\sigma_{all} = 0.6Fy = 0.6(50{,}000) = 30{,}000$ psi

And the strength of the section has been exceeded by a factor of 3.

However the validity of this result rests on the initial assumptions of complete fixity (rigidity) of the support, and lack of any restraining force on the opposite side of the member.

Further, this is in reality a complex, 3 dimensional bending problem which cannot be adequately analysed using simple statics. Because a moment is applied to a curved member, compressure forces normal to the plane of bending complicate the analysis.

Design Notes   Tunnel Project

The assumption of a fixed end is far from the actual case. In reality, the support will tend to rotate and act as a pin (i.e. 0 moment developed) Also, the interaction of the lagging/beam is not a rigid connection and therefore the system should deform to a state of zero moment. Further, the WF stiffening rib will generally be supported in the opposite direction, either by lagging (case previously analysed) or by the jacking system for advancement of the shield.

Design of Lagging

Must also consider the strength of the wood lagging to be used in the temporary lining of the tunnel.

Information for Design

Lagging - all 4"x6" spruce

Jacking Pressures 1)Maximum Pressure = 3500 tons
for shield/mole   2)Anticipated operating range 500-1000 tons

(Personal Communication, G. Emanuel, 1976)

Diameter of excavated tunnel = 20.5 feet

Thickness of lagging in tunnel = 4" = 0.333 ft

Area of annular ring from 20.5' to 20.16667 ft is

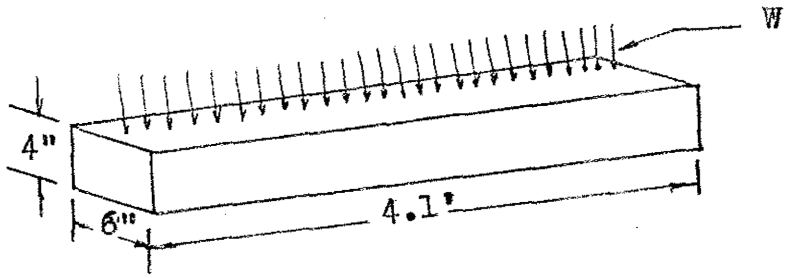
$$\text{Area} = \pi\left(\frac{20.5}{2}\right)^2 - \pi\left(\frac{20.5-0.3333}{2}\right)^2$$
$$= 10.7 ft^2$$
$$= 1533 in^2$$

Therefore, jacking pressure on timber lagging is

$$\text{Maximum} = \frac{3500*2000}{1533} = 4.57 \text{ ksi}$$
$$\text{Maximum Operating} = \frac{1000*2000}{1533} = 1.31 \text{ ksi}$$
$$\text{Minimum Operating} = \frac{500*2000}{1533} = 0.65 \text{ ksi}$$

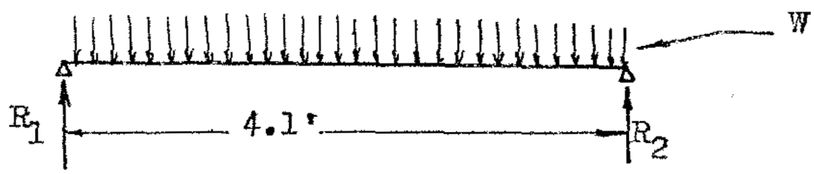
Design Notes   Tunnel Project

Now consider the design of the lagging as a simply supported beam.



First consider the long-term condition where no axial loads are applied to the lagging from the forward jacking of the shield.

Long-Term Case (simple bending of member)



$w_z = (K_o \gamma z)\ \frac{1}{2} = 1(5260)\ \frac{1}{2} = 2630^{\#}/\text{ft of lagging} = 219^{\#}/\text{in lagging}$

since exposed lagging section is 1/2' wide.

Therefore $R_1 = R_2 = \frac{wL}{2} = \frac{2630*4.1}{2} = 5400^{\#}$

S = section modulus $= \frac{bh^2}{6}$ for rectangular section

Therefore $S = \frac{6(4)^2}{6} = 16\ \text{in}^3$

$M_{max} = \frac{w\ell^2}{8} = \frac{219(4.1*12)^2}{8} = 66300\ \text{in}^{\#}$

<u>Design Notes</u> <u>Tunnel Project</u>

From page 52 Timber Design Manual (1972)

$$S_{req'd} = \frac{M\ K_m}{F'_b} \quad \text{where } K_m = \text{Moment Factor} = 1.0$$

Therefore $$S_{req'd} = \frac{M}{F'_b} = \frac{66300}{F'_b}$$

For sawn timber $F'_b = F_b K_{S_b} K_F K_D$

where K_{S_b} = service condition factor for bending = 1.0

K_F = treatment factor =1.0 for untreated wood

K_D = load duration factor = 1.0

and F_b = 1150 psi

Therefore $$S_{req'd} = \frac{66300}{1150} = 57.7 \text{ in}^3$$

Therefore, the required section modulus is much greater than the actual section modulus and failure of the lagging in the long-term condition can be expected.

Further, if axial loads due to the applied jacking force are considered, the situation is aggrevated.

Design Notes   Tunnel Project

Using the previous analysis, substantial lagging sectional properties are required in order to ensure the stability of the timber lagging. However, this result is grossly inconsistent with the current practice and performance of tunnelling carried out in the City of Edmonton.

The major problem involved in this portion of the analysis is the lack of capability to satisfactorily quantify the soil/structure interaction and to separate out the roles of the lagging and stiffening rib system.

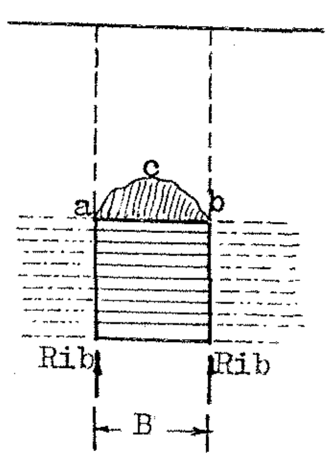
The theory of arching in soils when applied to the given boundary conditions leads to reasonable results.

Previously, the rib lagging system was assumed flexible when considering the design of the stiffening ribs. However, the ratio of the moduli of elasticity for both the wood lagging and the steel ribs is:

$$\frac{E_{wood}}{E_{steel}} = \frac{1}{25}$$

and in fact the wood lagging is substantially more ductile than the steel ribs.

Now, consider an individual cell in the tunnel consisting of two "stiff" steel ribs and the lagging



Terzaghi (1943) has outlined the theory of arching in soils. Laboratory investigations have shown that the pressure on a yielding strip such as a-b is almost independent of the state of stress existing in the soil at a height of more than 2B to 3B above the top of the strip.

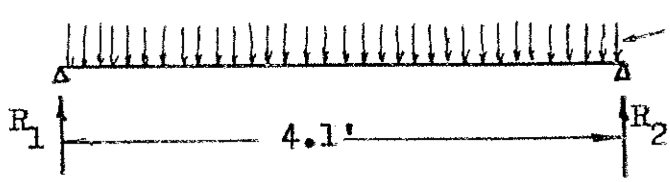
Further, Terzaghi and Peck (1967) indicate that the ultimate pressure imposed on a yielding strip is nearly totally independent of depth z of the tunnel below the ground surface. Further, the pressure exerted on the strip is approximately equivalent to the weight of the soil in the shaded area abc on the previous figure.

If this shaded area is assumed to be a rectangle having dimensions Bx2.5B (4.1'x2.5*4.1'), the estimated stresses acting on the lagging supported between the two steel stiffening ribs can be calculated. Therefore, assuming arching is taking place between the individual ribs, the earth pressure acting on the lagging at the top ℄ of the tunnel is equivalent to:

(4.1*10.25')

or a layer of soil 10.25' thick resting on the lagging.

Consider the lagging as a simply supported beam under the influence of the given pressure distribution (again lagging is 6" wide)



$W=\frac{1}{2}(10.25*135)$

$=692^{\#}/\text{ft.}/\frac{1}{2}$ ft. of wall

$$R_1=R_2=\frac{692*4.1}{2}=1420^{\#}$$

$w = 692^{\#}$/ft/1/2 ft of wall $= 58^{\#}$/in/1/2 ft of wall

Design Notes   Tunnel Project

Long-Term Condition

$$M_{max} = \frac{w\ell^2}{8} = \frac{58(4.1*12)^2}{8} = 17550 \text{ in lbs.}$$

From the Timber Design Manual (1972)

$$S_{req'd} = \frac{MK_m}{F'_b} \quad \text{where } K_m = 10 \quad \text{(defined previously)}$$

$$F'_b = 1150 \text{ psi}$$

Therefore, $S_{req'd} = \frac{17550*1.0}{1150} = 15.3 \text{ in}^3$

but $S_{actual} = 16 \text{ in}^3$ therefore O.K.

For shear in the member:

$$A_{req'd} = \frac{1.5V\ K_N}{F'_v} \quad \text{where } K_N = \text{notch factor} = 1.0$$

$$V = \frac{W}{2}\left(1 - \frac{2d}{L}\right) = \frac{2837}{2}\left(1 - \frac{2(4)}{49.2}\right) = 1188*$$

and F'_v for sawn lumber is:

$$F'_v = F_v K_{sv} K_F K_D$$

K_D = duration factor = 1.0

K_F = treatment factor = 1.0

K_{SV} = service condition for horizontal shear = 1.0

F_V = 85 psi (L.T.I.C., 1972)

Therefore, $A_{req'd} = \frac{1.5(1188)(1)}{85} = 21 \text{ in}^2$

but $A_{actual} = 24 \text{ in}^2$ therefore O.K. in shear.

The anticipated deflection of the member under long-term conditions is:

$$\Delta = \frac{5wL^4}{384E'I} K_\Delta$$

K_Δ = deflection factor = 1.0

$I = \frac{bh^3}{12} = \frac{6*4^3}{12} = 32 \text{ in}^4$

where $E' = EK_{SE}K_F$

where $K_F = 1.0$

$$\Delta = \frac{5(58)(4.1*12)^4}{384*1,200,000*32} = 0.12 \text{ inches}$$

K_{SE} = service condition factor = 1.0

E = 1,200,000 psi

Thus, in the long-term conditions a 4"x6" lagging member is sufficient. It should be pointed out that since the lagging is restrained against buckling in 3 directions, the unsupported length is equivalent to the length of the member.

Short-Term Design of Lagging

In the short-term the lagging/support system in the tunnel will be subjected to axial forces due to the jacking pressures of the mole and shield. It is necessary to evaluate this condition in order to assess the overall performance of the lagging. The anticipated jacking pressures have been outlined in a previous section.

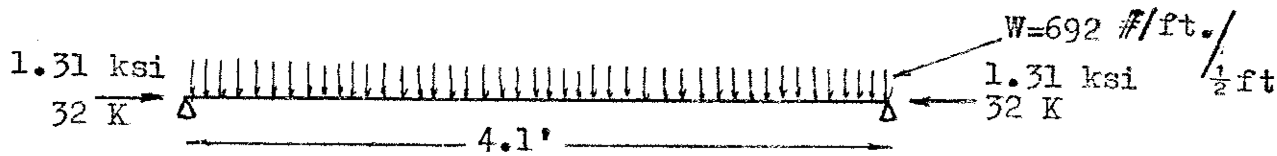
A member under the influence of combined bending and axial load must be designed to satisfy the following condition:

$$\frac{M/S}{F'_b} + \frac{P/A_N}{F'_a} \leq 1$$

where M = bending moment
S = section modulus
F'_b = allowable working stress in bending
P = axial load
A_N = net area
F'_a = allowable direct working stress

Design Notes Tunnel Project

Consider the performance of the lagging under the maximum anticipated operating pressure of 1.31 ksi:



$$W = 692*4.1' = 2837^{\#}$$

L = 49.2 inches
P = 32000#
I = 32 in^4
E = 1,200,000 psi

$$\Delta = \frac{5WL^3}{384EI} * \frac{1}{1 - \frac{PL^2}{10EI}}$$

$$= \frac{5(2837)(49.2)^3}{384(1,200,000)(32)} * \frac{1}{1 - \frac{32000(49.2)^2}{10(1,200,000)(32)}} = 0.115 * 1.253 = 0.144 \text{ inches}$$

$$M = \frac{WL}{8} + P\Delta = \frac{2837*49.2}{8} + 32000(0.144)$$

$$= 17550 + 4608 = 22158 \text{ in}^{\#}$$

$S = 16$ in^3

$P = 32000^{\#}$

$A_N = 24$ in^2

$F'_b = F_b K_{S_b} K_F K_D$ where F_b = 1150 psi

K_{S_b} = service condition factor for bending - 1.0

K_F = treatment factor = 1.0

K_D = load duration factor = 2.0 since jacking pressure can be considered an instantaneous load

therefore F'_b = 2300 psi

$F'_a = F_c K_{sc} K_F K_D K_C$ where F_c = allowable unit stress parallel to grain = 800 psi

K_{sc} = service condition factor for compression parallel to grain =0.91

K_F = treatment factor = 1.0

K_D = load duration factor = 2.0

K_C = slenderness factor = 1.0

Therefore $F'_a = 800(0.91)(1.0)(2.0)(1.0) = 1450$ psi.

Therefore $$\frac{M/S}{F'_b} + \frac{P/A_N}{F'_a} = \frac{\frac{22150}{16}}{2300} + \frac{\frac{32000}{24}}{1450} = 0.60 + 0.92$$

$$= 1.52 \not\leq 1.0$$

Therefore the stresses at the maximum anticipated operating pressure exceed the allowable stresses.

At an operating pressure of 500 Tons

$$\Delta = 0.115 * \frac{1}{1 - \frac{16000(49.2)^2}{10(1200,000)(32)}} = 0.129 \text{ inches}$$

Therefore $M = \frac{wL}{8} + P\Delta = 17550 + 16000(0.129) = 19600 \text{ in}^{\#}$

and $$\frac{M/S}{F'_b} + \frac{P/A_N}{F'_a} \leq 1 \Rightarrow \frac{\frac{19600}{16}}{2300} + \frac{\frac{16000}{24}}{1450} = 0.53 + 0.46 = 0.99 \simeq 1.0$$

Therefore at the lower anticipated operating pressure the stresses within the lagging are at the design limit.

Design Notes   Tunnel Project

At an operating pressure of 3500 Tons

$$\Delta = 0.115 * \frac{1}{1 - \frac{109000*(49.2)^2}{10(1200000)(32)}} = 0.37 \text{ inches}$$

Therefore $M_{max} = \frac{wL}{8} + P\Delta = 17550 + 109000(0.37) = 57900$ in lbs

and $\frac{M/S}{F'_b} + \frac{P/A_N}{F'_a} = \frac{\frac{57900}{16}}{2300} + \frac{\frac{109000}{24}}{1450} = 4.70 \nleq 1$

and the design criterion is again exceeded.

From the previous consideration of the lagging in the short-term case, under the influence of the jacking pressure of the mole and shield, in all cases the stresses in the lagging members are at or above the design limit.

However, the recommended stress levels in bending and axial load suggested in the Timber Design Manual (1972) are conservative. Dr. J. Longworth, Department of Civil Engineering U. of A. (Personal Communication 1976) indicated that on a statistical basis, the stresses at failure for a timber member are at least twice as large as the recommended design values (i.e. Factor of Safety of 2).

It is therefore anticipated that the lagging will perform satisfactorily in the short-term case when the applied jacking loads are within the range of anticipated working loads of 500 to 1000 tons.

However, if the jacking load required to advance the mole and shield approaches the maximum capacity of the jacks, i.e. 3500 tons, failure of the lagging immediately adjacent to the tunnelling machine can be expected.

At larger distances from the tunnelling machine the applied axial loads due to jacking will be some fraction of the actual jacking loads due to the soil/structure interaction. Some of the load will be dispersed as a shear stress development between the lagging and the soil. The actual distribution is difficult to quantify however.

Design based on the critical section adjacent to the tunnelling machine can be considered as the limiting case.

As well, some flexure will be introduced into the lagging members as they pass from the shield into contact with the soil.

Settlement above Tunnel

It has been found,through instrumentation and observation of several tunnels, that the settlement profile above a single tunnel may be reasonably represented by the normal distribution curve or error function.

In order to estimate the settlement profile above a tunnel the maximum settlement,δmax, must be known or assumed. Assumptions regarding the maximum settlement that may take place can be made from observations of the performance of other tunnels under similar soil conditions. Further, the width of the settlement trough must be known.

From observations and recorded data a relationship between the width of the settlement trough and the dimensionless depth of a tunnel has been developed for various tunnels passing through different materials (Peck 1969). This relationship is shown in Figure 5.

Design Notes Tunnel Project

An approach to estimating the settlements above a tunnel, is to assume a certain percentage of lost ground (a percentage of the tunnel area over a specified length) and calculate the settlement profiles for various quantities of lost ground.

The area of the trough can be estimated using the relationship:

$$\text{Area} = 2.5\, i\, \delta max$$

or

$$\text{Volume} = 2.5\, i\, \delta max \text{ per unit length of tunnel.}$$

For the proposed tunnel:

Z = depth of ℄ below ground surface = 40'

R = radius of tunnel = 20'

Therefore $\frac{Z}{2R} = \frac{40}{20} = 2$

and for soft to stiff clays from the relationship plotted on Figure 5 we have:

$$\frac{i}{R} = 1.5 \text{ therefore } i = 15'$$

Now consider the settlement profiles for lost ground amounting to 1,2,3,4,...% of the tunnel area.

Initial diameter = 20.5 ft Area = 330 ft^2

% Lost Ground	Volume of soil (Area/unit length of tunnel)	δmax (ft)
1	3.3 ft^2	0.088
2	6.6	0.176
3	9.9	0.264
4	13.2	0.352
5	16.5	0.440
7	23.1	0.616
10	33.0	0.880
15	49.5	1.32

from $\delta max = \frac{V}{2.5i} = \frac{V}{2.5(15)} = \frac{V}{37.5}$

TABLE I

DISTANCE FROM ℄ (ft)	DISTANCE / i	FRACTION OF δ_{max} (ft)	ORDINATE OF SETTLEMENT DISTRIBUTION (feet) 1%	2%	3%	4%	5%	7%	10%	15%
0	0	1	0.088	0.176	0.264	0.352	0.440	0.616	0.880	1.32
± 5	0.33	0.95	0.083	0.167	0.251	0.334	0.418	0.585	0.836	1.25
±10	0.67	0.80	0.070	0.141	0.211	0.282	0.352	0.493	0.704	1.06
±15	1.00	0.61	0.054	0.107	0.161	0.215	0.268	0.376	0.537	0.805
±20	1.33	0.41	0.036	0.072	0.108	0.144	0.180	0.253	0.361	0.541
±25	1.67	0.25	0.022	0.044	0.066	0.088	0.110	0.154	0.220	0.330
±30	2.00	0.14	0.012	0.025	0.037	0.049	0.062	0.086	0.123	0.185
±35	2.33	0.07	0.006	0.012	0.018	0.024	0.031	0.043	0.062	0.092

The calculations for the different assumed values of lost ground are summarized in Table 1.

The different ordinate values were calculated using the properties of the normal distribution curve (Neville and Kennedy, 1964).

Settlements above Tunnel Pairs

The settlement occurring above a pair of tunnels can be estimated using one of the following approaches:

(a) Superposition of the settlements of the individual tunnels.

or (b) Determination of a settlement trough for a single tunnel using an equivalent radius based on the diameters of the single tunnel. The center line of this combined settlement rough will correspond with the line of symmetry of the two tunnels.

Approach A

From the previous calculations the effects of superposition of the individual settlement curves can be summarized.

It has been assumed that the tunnels are at 40' centre-to-centre for simplicity of calculations. The actual spacing is 39 feet centre-to-centre. The differences in the estimate resulting from this assumption are negligible.

The calculations are summarized in Table II.

Approach B

Using this approach an equivalent radius for the twin tunnels can be calculated from

$$R' = R + \frac{d}{2} \quad \text{where } d = \text{spacing (Peck, 1969)}$$

TABLE II

SETTLEMENT ABOVE DUAL TUNNELS

- APPROACH A - SUPERPOSITION

ORDINATES OF SETTLEMENT DISTRIBUTION

(ft)

FOR ASSUMED % LOSS OF GROUND

DISTANCE FROM ℄ (ft)	1%	2%	3%	4%	5%	7%	10%	15%
0	0.072	0.144	0.216	0.288	0.360	0.506	0.722	1.082
± 5	0.076	0.151	0.227	0.303	0.378	0.530	0.757	1.135
± 10	0.082	0.166	0.248	0.331	0.414	0.579	0.827	1.245
± 15	0.089	0.179	0.269	0.358	0.449	0.628	0.898	1.342
± 20	0.089	0.178	0.267	0.356	0.445	0.623	0.890	1.335
± 25	0.083	0.167	0.251	0.334	0.418	0.585	0.836	1.25
± 30	0.070	0.141	0.211	0.282	0.352	0.493	0.704	1.06
± 40	0.036	0.072	0.108	0.144	0.180	0.253	0.361	0.541
± 50	0.014	0.025	0.037	0.049	0.062	0.086	0.123	0.185
± 60	0.001	0.002	0.003	0.004	0.005	0.007	0.010	0.015

TABLE III

SETTLEMENT ABOVE TWIN TUNNELS

- APPROACH B - EQUIVALENT RADIUS = 30'

% LOST GROUND IN INDIVIDUAL TUNNELS			ORDINATES OF SETTLEMENT DISTRIBUTION (ft) FOR ASSUMED % LOSS OF GROUND							
			1%	2%	3%	4%	5%	7%	10%	15%
VOLUME OF SOIL (AREA/UNIT LENGTH)		yds^3	0.24	0.42	0.73	0.98	1.22	1.71	2.44	3.67
		ft^3	6.6	13.2	19.8	26.4	33.0	46.2	66.0	99.0
DISTANCE FROM ℄ (FEET)	DISTANCE / i	FRACTION OF δmax								
0	0	1	0.125	0.251	0.377	0.503	0.629	0.880	1.26	1.89
± 10'	0.476	0.891	0.111	0.224	0.336	0.448	0.560	0.784	1.12	1.68
± 20'	0.952	0.637	0.080	0.160	0.240	0.320	0.401	0.561	0.803	1.20
± 30'	1.429	0.360	0.045	0.090	0.136	0.181	0.226	0.317	0.454	0.680
± 40'	1.905	0.165	0.021	0.041	0.062	0.083	0.104	0.145	0.208	0.312
± 50'	2.381	0.059	0.007	0.015	0.022	0.030	0.037	0.052	0.074	0.112
± 60'	2.857	0.017	0.002	0.004	0.006	0.009	0.011	0.015	0.021	0.032

therefore $R' = 10 + \frac{40}{2} = 30'$

therefore $\frac{Z}{2R'} = \frac{40}{60} = 0.67 = \frac{i}{R'} = 0.70$ therefore $i = 21'$

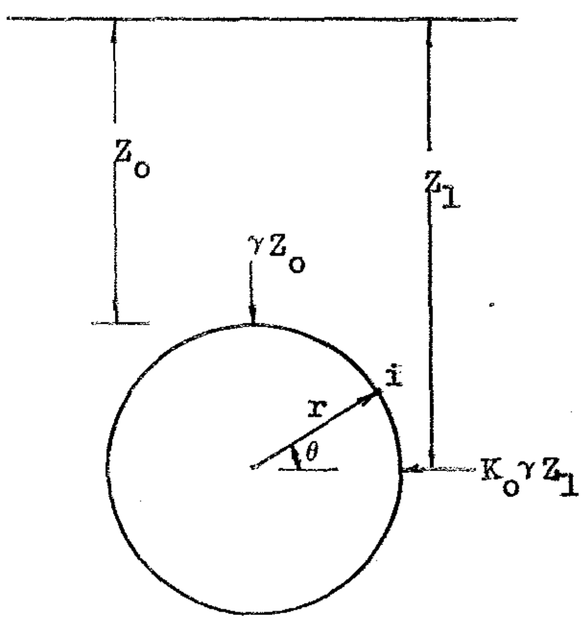
therefore $V = 2.5\ i\ \delta max$ per unit length of tunnel

therefore $V = 21(2.5)\ \delta max = 52.5\ \delta max$

The calculations are similar to those for a single tunnel and are summarized in Table III.

Finite Element Analysis

Assuming the original stress distribution outlined by Peck (1969), the nodal forces required to give a stress free boundary in the finite element analysis are calculated below.



Design Notes Tunnel Project

At point i

$$\sigma_1 = \gamma(z_1 - r \sin\theta)$$

$$\sigma_3 = K_o \gamma(z_1 - r \sin\theta)$$

$$\sigma_i = K_o \gamma(z_1 - r \sin\theta) \cos^2\theta + \gamma(z_1 - r \sin\theta) \sin^2\theta$$

$$\sigma_i = \gamma(z_1 - r \sin\theta) [K_o(1 - \sin^2\theta) + \sin^2\theta]$$

$$\sigma_i = \gamma(z_1 - r \sin\theta) [K_o + \sin^2\theta (1 - K_o)]$$

Calculating stress relief that takes place upon excavation of the tunnel.

Know:

$$\sigma_i = \gamma(z_1 - r \sin\theta) [K_o + \sin^2\theta (1 - K_o)]$$

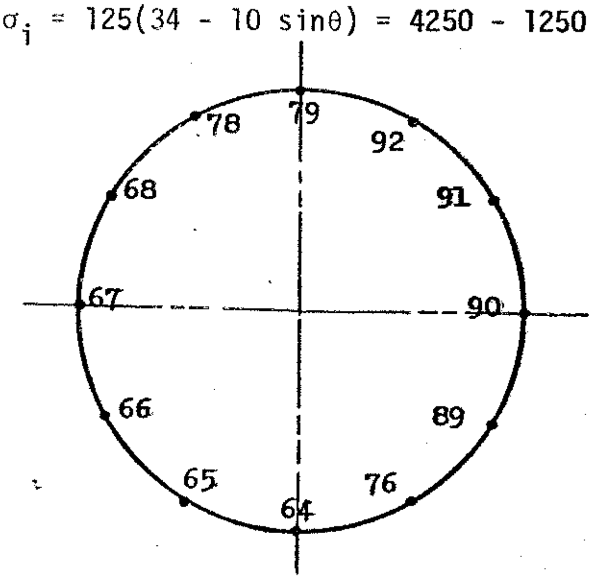
γ = 125 p.c.f.

K_o = 1.0

z_1 = 34

r = 10 ft

$$\sigma_i = 125(34 - 10 \sin\theta) = 4250 - 1250 \sin\theta$$



Design Notes Tunnel Project

for nodes 90 and 67

$$R_{90} = r \int_{-15°}^{15°} = (4250 - 1250 \sin\theta)d\theta = 42500\ \theta + 12500 \cos\theta \Big|_{-15°}^{+15°}$$

$$R_{90,67} = 42500\ (0.2618 + 0.2618) = 22{,}253 \text{ lbs}$$

for nodes 91,68

$$R_{91} = 10 \int_{15°}^{45°} (4250 - 1250 \sin\theta)d\theta = 4250 * 0.5236 + 12500\ (0.707 - 0.966)$$

$$R_{91,68} = 19{,}016 \text{ lbs}$$

Node 92,78

$$R_{92} = 22253 + 12500\ (0.259 - 0.707) = 16649 \text{ lbs}$$

Node 79

$$R_{79} = 22253 + 12500\ (-0.259 - 0.259) = 15783 \text{ lbs}$$

Node 89,66

$$R_{89} = 22253 + 12500\ (0.966 - 0.707) = 25{,}940$$

Design Notes Tunnel Project

Node 76,65

$R_{76} = 27853$

$R_{64} = 28723$

Node	Angle From Horizontal	R(lbs)	Rx(lbs)	Ry(lbs)
64	90	28723	0	28723
65	60	27853	13927	24121
66	30	25940	22465	12970
67	0	22253	22253	0
68	30	19016	16468	-9508
76	60	27853	-13927	24121
78	60	16649	8325	-14419
79	90	15783	0	-15783
89	30	25940	-22465	12970
90	0	22253	-22253	0
91	30	19016	-16468	-9508
92	60	16649	-8325	-14419

Design Notes   Tunnel Project

Finite Element Analysis - Imposed Displacements

Another portion of the finite element analysis carried out in the previous investigation imposed a radial displacement of 2 inches inward on the tunnel. The x-y magnitudes of the imposed displacements are listed below.

Node	X-Displacement (ft)	Y-Displacement (ft)
79	0.0	-0.1667
92	-0.0833	-0.1443
91	-0.1443	-0.0833
90	-0.1667	0.0
89	-0.1443	0.0833
76	-0.0833	0.1443
64	0.0	0.1667
65	0.0833	0.1443
66	0.1443	0.0833
67	0.1667	0.0
68	0.1443	-0.0833
78	0.0833	-0.1443