A PROBABILISTIC APPROACH TO SOFT GROUND TUNNELING

by

ANDREW SLUZ

Submitted in partial fulfillment
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ABSTRACT

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The designer of soft ground tunnels often treats geologic data as deterministic and formulates his design accordingly. When uncertainty in ground conditions is recognized, it is recognized in an abstract manner and cannot be used objectively to help in the design process. Often the solution to tunneling difficulties due to geologic uncertainty is relegated to the construction phase of the process, to be resolved when encountered. Such treatment of the probabilistic nature of geology leads to uneconomical designs and an increase in the cost of soft ground tunneling. In urban areas tunnels such as those for mass transportation are of large diameter and are near the surface. Here the difficulties of tunneling are magnified by the adverse impact poor tunnel construction planning can have on surface structures.

This thesis presents an approach to the design of soft ground tunnels which requires that construction planning be made part of the design process and that the designer quantify his subjective notions of the variability of the ground conditions to be encountered. Subjective degree of belief probability is used to evaluate the possible occurrence of alternate states of the ground parameters that the engineer determines have a significant effect on tunnel cost. A set of geologic parameters are presented as one possible configuration of the probabilistic model of geology for soft ground tunneling in the urban environment. The parameters are structured into a decision tree framework similar to that used by Vick (39) to model hard-rock geology.

A general tunnel cost model is formulated to assist in evaluating the financial consequences of variation in individual geologic parameters. Utilizing this general cost model, the economic relevance of the occurrence of alternate states of geologic parameters can be evaluated. Equivalent monetary values are

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then used as indices to evaluate the relative suitability of alternate tunnel construction system designs.

The proposed approach is a method for structuring the design process so that geologic uncertainties can be evaluated in a uniform fashion. The general cost model and the probabilistic, decision tree geologic models are tools which the designer can use to evaluate design alternatives to arrive at a tunnel/construction system design which is most likely to be economical and successful for a given set of soft ground geologic data.

Thesis Supervisor: Dr. Herbert H. Einstein

Title: Associate Professor of Civil Engineering

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I also cannot thank nearly enough my dear wife, Muriel for the pregnant hours she spent typing this thesis, coping simultaneously with my broken handwriting and English.

TABLE OF CONTENTS

LIST OF SYMBOLS AND KEY ABBREVIATIONS

P[C] . . . . the probability of the occurrence of state C

$\cap$ . . . . . the intersection of random parameter states, e.g. $A \cap B$ is the occurrence of random states A and B

$\cup$ . . . . . the union of random parameter states, e.g. $A \cup B$ is the occurrence of random states A or B

a. . . . . . tunnel radius

bemv(n). . . branch equivalent monetary value for n^{th} state of a given parameter

emv. . . . . equivalent monetary value summed over one parameter

c. . . . . . soil shear strength

ℓ. . . . . . tunnel segment length

p_i . . . . . tunnel internal pressure

p_z . . . . . total vertical stress at depth z

r_j . . . . . rate of shield advance

s_u . . . . . undrained shear strength

t_a . . . . . support erection time

t_s . . . . . ground standup time

z. . . . . . tunnel depth

A. . . . . . ground identification parameter

A_i . . . . . i^{th} component of ground invariable tunnel cost

B. . . . . . gas occurrence parameter

B_i . . . . . i^{th} component of ground variable tunnel cost

C. . . . . . water inflow parameter

D. . . . . . stability parameter

E. . . . . . long term structural adequacy parameter

F. . . . . . additional support of structures and utilities parameter

G. . . . . . boulder and obstacle occurrence parameter

D_{10}. . . . . effective grain size parameter

D_r . . . . . relative density

EMV. . . . . total equivalent monetary value for whole decision tree

K_n . . . . . n^{th} end branch cost consequence

Ko . . . . . coefficient of lateral earth pressure at rest

OCR. . . . . overconsolidation ratio

R. . . . . . radius of plastic zone about tunnel

R_s . . . . . unit standup time for a one foot wide strip of unsupported tunnel roof

ROA. . . . . average tunnel construction rate of advance

SR . . . . . stability ratio

STR. . . . . standup time ratio (t_s/t_a)

T. . . . . . total tunnel project cost

T_ℓ . . . . . cost of tunnel segment of length ℓ

$T_{\ell a}$. . . . . cost of tunnel segment of length ℓ subject to ground parameters represented by decision tree branch with end node probability a

To, To_ℓ. . . non-ground variable cost of tunnel project and segment of length ℓ respectively

U. . . . . . uniformity coefficient

V_s . . . . . volume of settlement trough

X_n . . . . . cost factor, ground variable material costs

Y. . . . . . labor cost rate

Z. . . . . . operating cost rate

$\alpha[C]$ . . . . geologic parameter whose state is C

∇. . . . . . mass parameter of α

γ. . . . . . unit weight

δ. . . . . . surface settlement

ζ. . . . . . relative contribution factor

κ. . . . . . nodal cost consequence

ξ_n . . . . . n^{th} ground variable cost element

σ_1 . . . . . minor principal stress

σ_3 . . . . . major principal stress

ϕ. . . . . . angle of internal friction

ϕ_r . . . . . angle of repose

INTRODUCTION

1.1 Purpose and Scope

The construction of mass transportation guideway systems beneath urban areas is an attractive alternative to either surface or elevated guideway construction. Transit systems in tunnels leave the surface free to be used for other, more attractive or important needs: and impact the environment far less, especially during construction. Unfortunately tunnels are also more expensive to build than surface or elevated guideway structures. Some of the additional cost for construction underground instead of on the surface is due to the inability of the designer to predict exactly the ground conditions which will be encountered by the tunnel alignment. This geologic uncertainty must somehow be resolved so that the loads imposed by the ground on the tunnel structure can be evaluated in the design.

Another difficulty of tunneling is that the occurrence of unexpected deviations in geologic conditions can hamper tunnel construction; because the construction system was not suitable to deal with the particular ground conditions that arose. Often the engineer may suspect the possibility that variations in predicted geologic conditions could occur; but his inability to quantify his suspicions leads to overly conservative designs or cost estimates. The need for further site investigation to resolve geologic uncertainties can also be difficult to justify because often the results of the investigations serve only to modify the engineer's subjective opinions of what the ground conditions really are, instead of definitively resolving the uncertainty.

This thesis presents an approach to the design of soft ground tunnels in urban areas which takes into account the probabilistic nature of the ground. Subjective degree of belief probability is employed as a means of quantifying the engineer's assessment of geologic uncertainty. A decision tree geologic model, such as that used by Vick (39) for modeling hard rock geology, is presented with ground parameter states typical of tunneling in the urban environment. These ground parameters are behavioristic and therefore depend upon the type of construction system employed; necessitating that tunnel design encompass construction planning and structural design simultaneously. Equivalent monetary values are used as criteria for decision-making. These tools are further defined and their application amplified in the remainder of this introductory chapter.

1.2 Current Approaches to Tunneling - The Problem

Urban mass transportation systems are usually planned in advance of detailed engineering design phases. Estimates are based on preliminary engineering data including geologic mapping and some soil boring which is correlated with available geologic data. Emphasis in site investigations is on the classification of materials first and then on applying very broad, behavioral generalizations such as "loose" or "soft" to the properties of encountered materials. Such analyses usually set horizontal and vertical alignments by fixing the end points of the routes, allowing very little variation in successive stages.

The design/construct phase is separated into two distinct parts with only the plans and specifications as a connecting link. The first phase is accomplished by the owner's engineer and consists of steps which usually follow this format:

1) site investigation
2) preliminary design and estimate
3) evaluation of all design components
4) additional site investigation based on step 3
5) final design and engineer's estimate

The project is advertised for bids and the contractor then takes the active role while the owner's engineer is passive. The goal of the contractor is to submit a successful bid, which is defined as the highest bid possible which will be lower than those of his competitors, yet still allow him to make a suitable profit. His approach may resemble the following:

1) cursory examination of site information

with little or no independent exploration

2) modeling of construction system

3) estimation of costs

4) reevaluation of bid according to job desirability

5) bid submission

Often any time spent by the contractor for further site investigation is for the purpose of refining the knowledge of ground conditions with respect to a single tunnel cost element, such as the cost of tunnel liner or ground stabilization, where the contractor feels he may either widen his profit margin or reduce his bid price through changes in the element costs. The evidence from further site investigation is used to help decide what the final bid will be, such evidence can help in the assessment of risk that is involved with the submission of a particular bid. The bid itself is a reflection of the model of construction and ground which the contractor has assumed and therefore is subject to the uncertainty of both the ground conditions and the interaction of construction with these ground conditions, just as in the case of the design. Now the true inequity is exposed, for even if the uncertainties of ground conditions could be completely resolved the division of design and construction into individual parts, which are the responsibility of separate parties, may allow the design and the planning of the construction to be performed under different assumptions of ground behavior which may be in conflict once actual construction begins. Any logical approach must then accomodate the whole tunnel system, the configuration of the tunnel elements (the design,) and the methods employed in accomplishing them (the construction.)

The combination of tunnel design and construction planning into one step allows the formulation of a single model of the ground which eliminates the possibility of conflicting assumptions of ground behavior as a factor in the success of the tunneling process. The analysis of the uncertainty with respect to ground conditions alone now has more meaning because it can be identified with only one model of ground behavior and decisions can be made by quantifying these uncertainties and analyzing their effect on the tunneling process. From this point forward then, tunnel design will be assumed to include construction planning.

1.3 Modeling Tunnels and Geology - A Solution

To resolve the effect that the uncertainty with regard to ground conditions has on the tunnel design, an approach is postulated which incorporates a probabilistic evaluation of geology into the design process. The foundation of this approach is that an economy of design can be achieved if the designer subjectively quantifies his conception of the ground to be tunneled and evaluates his design within the framework of this probabilistic interpretation of geology. If the tunnel construction can be modeled through the use of suitable parameters which are sensitive to variations in the ground, then an iterative approach, as shown in Figure 1.1 can be used to arrive at a design which is in harmony with its environment. The remainder of this section will introduce the general aspects of the tunnel systems model and the framework for structuring the geologic model. Then, an approach to evaluating the interaction between tunnel and ground will be presented which will be the final concept necessary for an understanding of the general approach suggested in Figure 1.1 (specific details for applying it will be presented later.)

1.3.1 Tunnel Model Framework

"Model" is really a word used to represent the terms by which an event or process is described. These terms can be related to each other and summed to describe the complete action of the process of interest, as each term varies as a function of some outside influence. A simple example of such a model is the following equation:

$$T(\$) = X(\$/LF) \cdot L(LF) \qquad [1.1]$$

which describes the total cost in dollars of a tunnel of length L as a function of a factor, X, which relates the

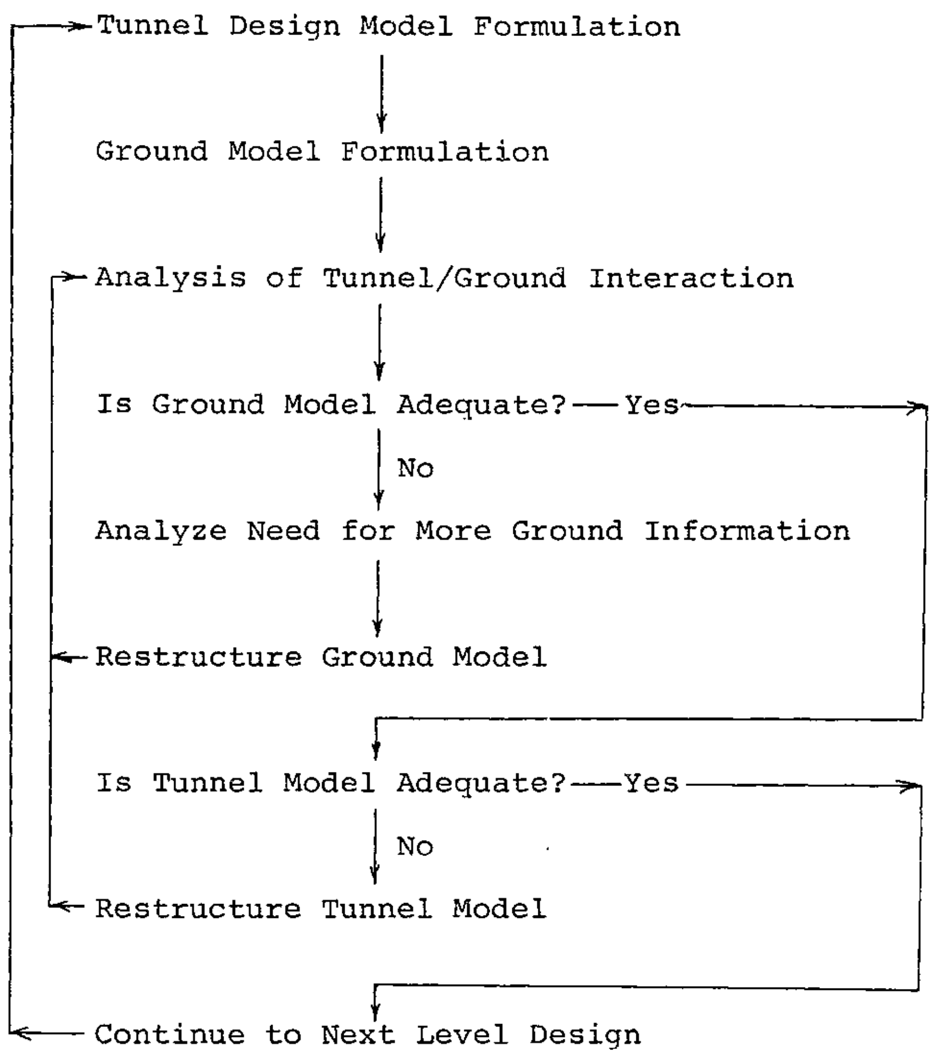


Figure 1.1. GENERAL TUNNEL DESIGN APPROACH

cost per unit length of a tunnel to some outside influence such as ground condition. The planner may have from previous experience two factors, $X_1 = A$ and $X_2 = B$, where X_1 is the unit cost of a particular design of tunnel in sand and X_2 the same tunnel design in clay. Now if the planner/designer were to compare another design to this one in a given ground condition he would need two other factors, X_3 and X_4, which are to this new design as X_1 and X_2 are to the first. The designer would have a model, sufficient for the level of detail dictated by his requirements, upon which to make a decision. Of course as the design is refined and the knowledge of the ground is increased this level of model is insufficient because there are too many factors which may be masked by variations in the total cost.

The next level of detail will break the total costs into a series of components, some of which are affected by changes in ground conditions and some of which are unaffected. This would be similar to saying that the total cost, T, is equal to the sum of the cost of all the components of the tunneling process which are unchanged when ground conditions vary, A, and of those components which do change with variation in ground condition, B. Expressed mathematically:

$$T = A + B \qquad [1.2]$$

where:

$$A = \sum_{i=1}^{n} A_i \qquad [1.2a]$$

$$B = \sum_{j=1}^{m} B_j \qquad [1.2b]$$

and the total cost is then:

$$T = \sum_{i=1}^{n} A_i + \sum_{j=1}^{m} B_j \qquad [1.2c]$$

where there are i ground invariable, and j ground variable tunnel cost elements.

The i components of cost element A, which remains unaffected by variations in the ground behavior, are no longer of interest and can be represented by a single fixed cost, T_0, such that:

$$T = T_0 + \sum_{j=1}^{m} B_j \qquad [1.2d]$$

It is not meant to imply that T_0 is non-variable, or that the variation in T_0 is not important relative to the success or failure of a project. This cost includes such items as liner prices which may not vary with ground conditions but will vary with the laws of supply and demand and is every bit as much subject to uncertainty at the time of design.

Spooner (33)* and Vergara and Boyer (38) apply the probabilistic approach to estimate the cost of those elements of T such as material prices and labor rates which can vary with non-ground related factors such as supply and geographic location of product. These cost elements can be approached from a relative frequency view of probability, where uncertainties can be quantified based on previous costs for these elements in similar situations. Such an analysis of total cost should be included in the design just for completeness and a better understanding of the relation of those cost variations which are due to ground conditions to the tunnel construction cost as a whole. The scope proposed here however covers design with respect to geology only and therefore such elements as labor costs will be assumed fixed unless the change in cost is due directly to ground conditions. Tunnel costs which do and do not vary with ground changes will be identified later.

The form of the cost model has been established, the components of the tunneling process will be described by their cost. This automatically imposes the condition on the geologic model that it must accept only parameters which have consequences that can be interpreted as costs. This is a standard engineering approach, though somewhat in dispute by the non-engineering world as a materialistic one, which

*Number in parentheses refers to Bibliography.

ignores the humanistic consequences of a project. The quantification of all tunneling aspects within identifiable parameters that can be used in mathematical analyses is central to the development of this approach and it therefore will be assumed that all construction consequences, including environmental, social, and geological, can be expressed in dollars.

1.3.2 Geologic Model Framework

The tunnel designer has always been faced with the problem that the model of geologic conditions which he uses as a tool in developing his design is not deterministic as is implied by the soils profile with which he works. This profile is, rather, an interpretation of what he feels is the most likely configuration of geology to be inferred from the evidence of site investigations. The random nature of the samples of underground conditions as obtained from boreholes, and the errors in testing for the material behavior of these samples is the source of the uncertainty with respect to ground conditions with which the designer is confronted. It is only natural to apply a probabilistic approach to this modeling, to account for these uncertainties. Subjective, degree-of-belief probability will be used to quantify the engineer's understanding of ground conditions at a given location, then these conditions will be structured in decision trees representing all the possible parameters that can affect the tunneling process. These decision trees will reflect the engineer's opinions of the state of ground along an alignment given the data from site investigations. Bayes' Theorem will be used to reassess probabilities as new information from additional site investigation is gathered. This whole approach is not new, it was applied by Vick (39) in relation to hard rock tunneling, but it is relatively unknown to tunneling engineers; therefore these concepts will be described in detail below.

If it is known for a certainty that one particular state, say C, exists at a specific location in the ground, then the probability of the occurrence of that state at that location is unity or $P[C] = 1$. The probability of another state, say D, where C and D are mutually exclusive, occurring at that same point is zero or $P[D] = 0$. If, however, the engineer knows that at that particular location the state which exists is either C or D then the union of C and D, or the probability of the state at that location being one of either C or D, is unity. Expressed mathematically, this is $P[C] \cup P[D] = 1$. This can be extended to include any number of mutually exclusive, exhaustive sets of states. For example if E and F were also states mutually exclusive of C and D, and that with C and D constituted the universe of conditions that could exist at a given location, then at that location:

$$P[C] \cup P[D] \cup P[E] \cup P[F] =$$
$$P[C] + P[D] + P[E] + P[F] = 1 \qquad [1.3]$$

If one were to express these states as branches on a decision tree, each state dependant on the existence of node A and the occurrence of each state represented by an end branch node a, b, c, or d; then the probability of arriving at end node a, b, c, and d given A is just the probability of occurrence of states C, D, E, and F, or a = P[C], b = P[D], c = P[E] and d = P[F]. This concept is illustrated in Figure 1.2. The summation of the end branch node probabilities emanating from node A is the union of a, b, c, and d, or P[C] + P[D] + P[E] + P[F] = 1.

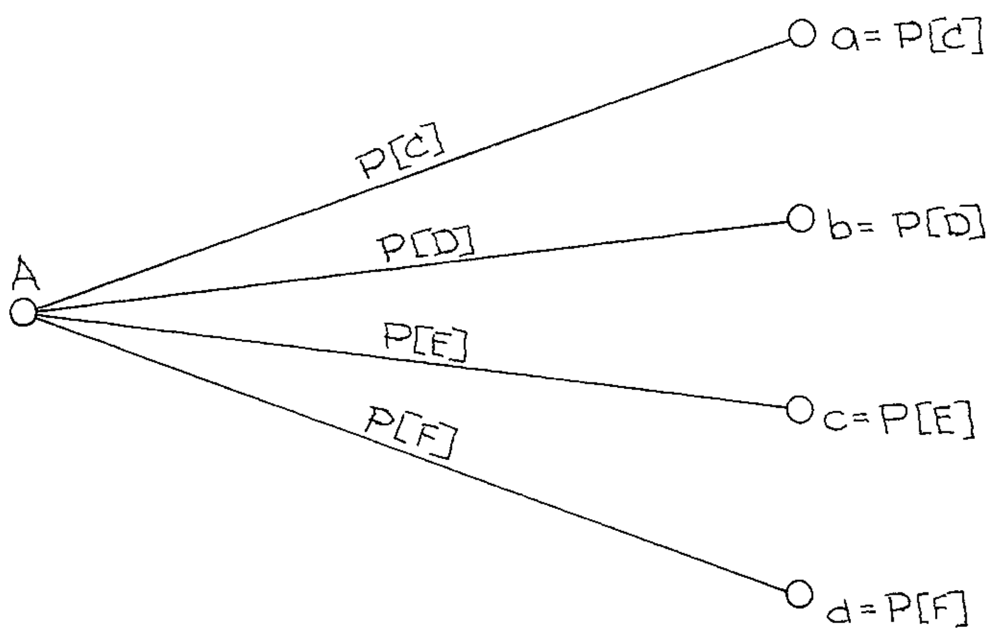


Figure 1.2. SAMPLE DECISION TREE

Consider now that for the engineer's purposes, the ground conditions at a given location could be described by two parameter sets α and β where the conditions C, D, E, and F were all the mutually exclusive and exhaustive states of α and similarly G and H were the mutually exclusive and exhaustive states of β. The occurrence of α and β at the location is strictly random so that the random occurrence of a particular state of α, α_0, and of β, β_0, at the same time is the intersection of α_0 and β_0, $\alpha_0 \cap \beta_0$, and is governed by an axiom of probability theory which states that the probability of two random events occurring at any one point is the product of the probabilities of each event occurring, or:

$$P[\alpha_0 \cap \beta_0] = P[\alpha_0] \cdot P[\beta_0] \qquad [1.4]$$

Applying this axiom to the quantification of uncertainty regarding ground conditions at a given point; the probability of the occurrence of a given end branch node representing one of the states of β, and of a particular state of α, will be computed using Equation 1.4. The new decision tree which represents all the alternate configurations of the state of ground at a point is as shown in Figure 1.3, with new end branch node probabilities a through h. Examining this decision tree, one can see that the union of states over any end branch nodes stemming from one node must be unity or, if P[A] = 1, then it must follow that:

$$a + b + c + d + e + f + g + h =$$

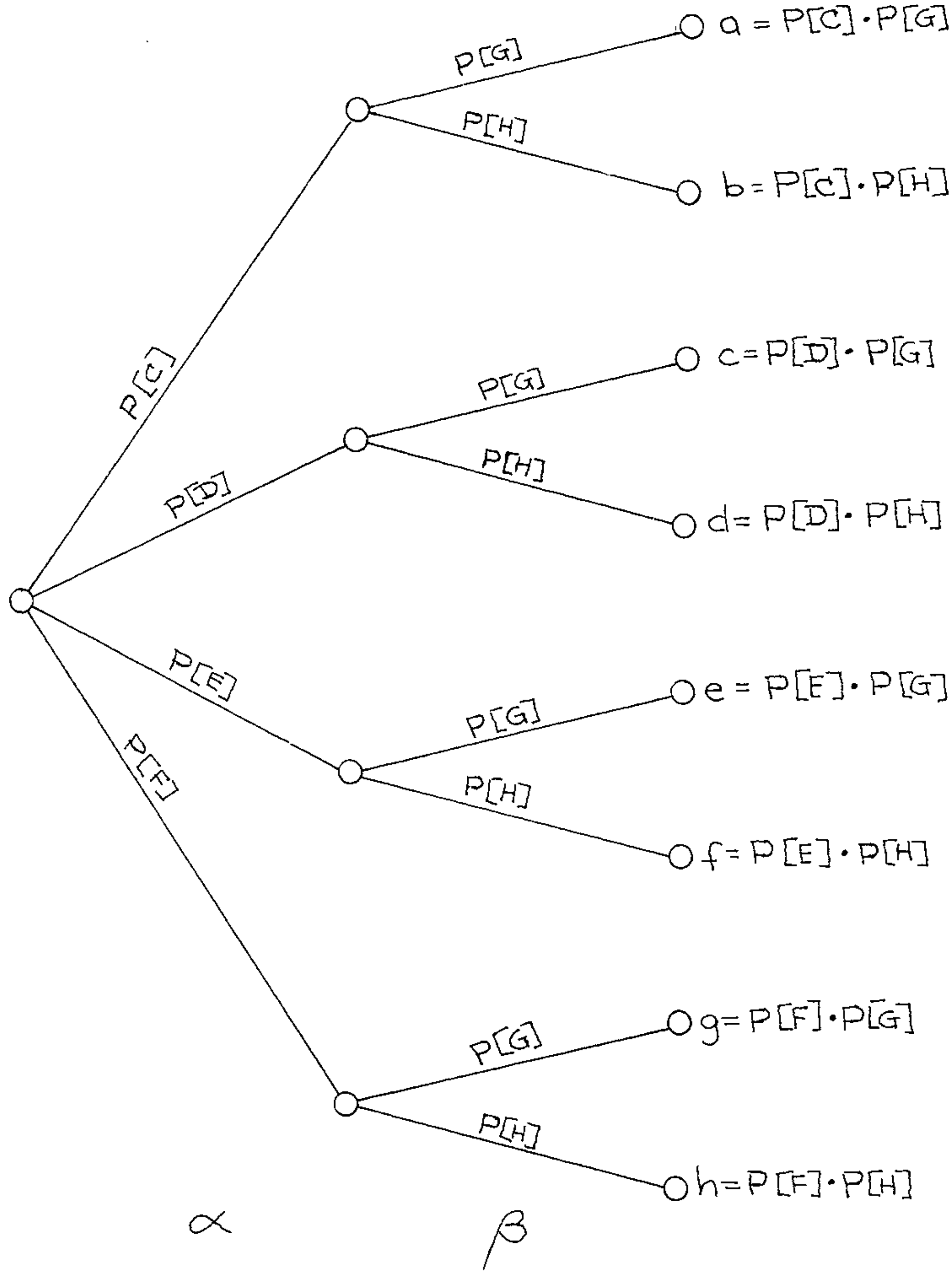


Figure 1.3 EXAMPLE OF DECISION TREE WITH TWO RANDOM STATES

$$P[C] \cdot P[G] + P[C] \cdot P[H] + P[D] \cdot P[G] +$$
$$P[D] \cdot P[H] + P[E] \cdot P[G] + P[E] \cdot P[H] +$$
$$P[F] \cdot P[G] + P[F] \cdot P[H] = 1.$$

If P[A] is not unity then the probability of any of the end branch nodes shown in Figure 1.3 occurring will be in the more general form, or: $a = A \cap C \cap G = P[A] \cdot P[C] \cdot P[G]$, $b = A \cap C \cap H = P[A] \cdot P[C] \cdot P[H]$, $c = \ldots.$.

a. Conditional Probabilities - Consider now the case where α and β are not wholly independent, but the occurrence of a particular state, β, is dependent on which state of α occurs. Say for example the designer knew that if α was either C or E [α(C or E,)] then β(G or H;) but if α(D) then β(G) and if α(F,) then β(H) followed. The decision tree could be structured to reflect these conditional probabilities as shown in Figure 1.4 and the end branch node probabilities, a through f, would be as shown given that when α(D,) $c = P[G] = 1$ and α(F,) $f = P[H] = 1$.

b. The Assessment of Probabilities, Bayes' Theorem - The engineer needs two general kinds of information included in his geologic model, an identification of materials along an alignment, in categories which imply general ranges of material behavior, to allow him to conceptualize general material behavior which is a function of many different soil properties such as effective grain size (D_{10}), plasticity, or compressibility. Specific details of the absolute varia-

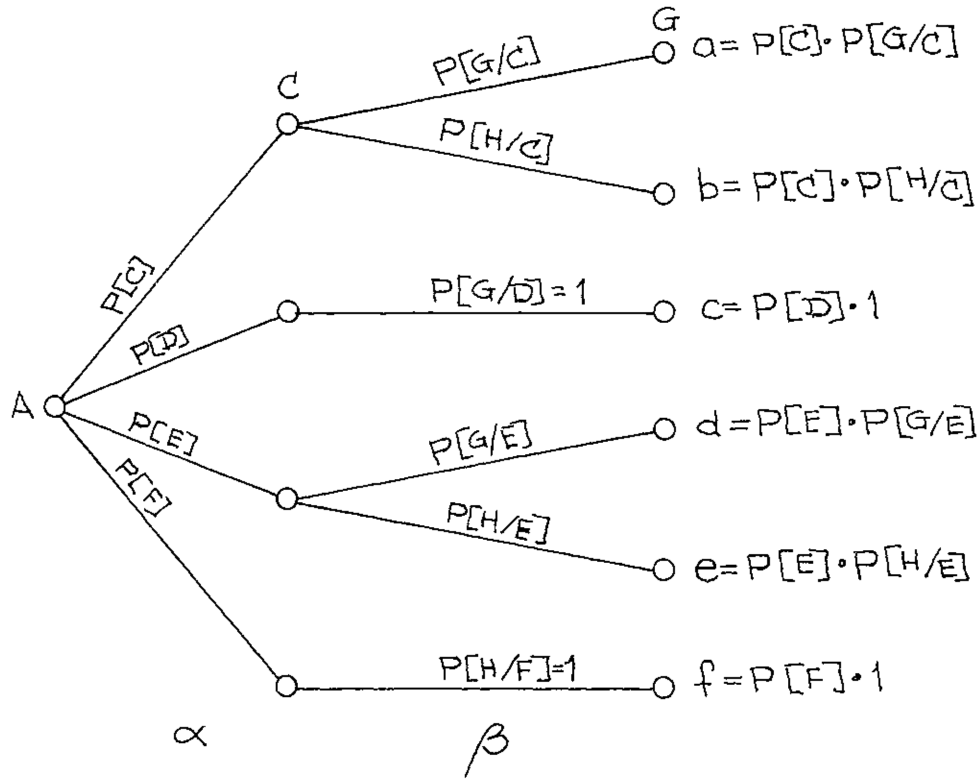


Figure 1.4 EXAMPLE OF DECISION TREE WITH TWO CONDITIONAL STATES

tions of material property values, such as strength or permeability, also are needed and have a major influence on the tunnel design. The soil profile, which the engineer uses as a tool to represent this information, is only a graphic illustration of what the engineer estimates to be the most probable distribution of soil conditions.

At borehole locations the probability of encountering the material identified by the sampling is close to unity.*

*There exists the possibility that the material straddles several classifications in which case specific tests may have to be applied to determine the degree of confidence the engineer has in classifying it one way or the other. In this case the discussion which follows on Baye's Theorem also applies to this problem of uncertainties.

Identifying specific characteristics of that material's behavior under a given loading condition or determining exact values of parameters which index the material behavior (as the strength and permeability may as mentioned in the previous paragraph) can be accomplished by the engineer only with a certain degree of confidence which is a function of such things as the engineer's previous experience with the material, his ability to interpret soils tests, and his particular predjudices with respect to the specific project he is working on.

When the point of interest with respect to ground conditions, say some point X, is not on a borehole, but is some distance away, the prediction of the geologic nature of X becomes more uncertain. If there are two boreholes equally distant from X each exhibiting mutually exclusive states of the same parameter α, say states C and D, at the elevation of X (as in Figure 1.5(a);) then the engineer may conclude that the chance of either state occurring at X is even, or P[C] = P[D.] The decision tree representative of the possible states of α, if the universe of mutually exclusive and exhaustive, states was C, D, E, and F, would be similar to the tree in Figure 1.2 with the same end branch node probabilities, a and b. If the engineer examined all the evidence of the boreholes, he could draw a hypothetical soils profile as illustrated by Figure 1.5(b) representing what, in the

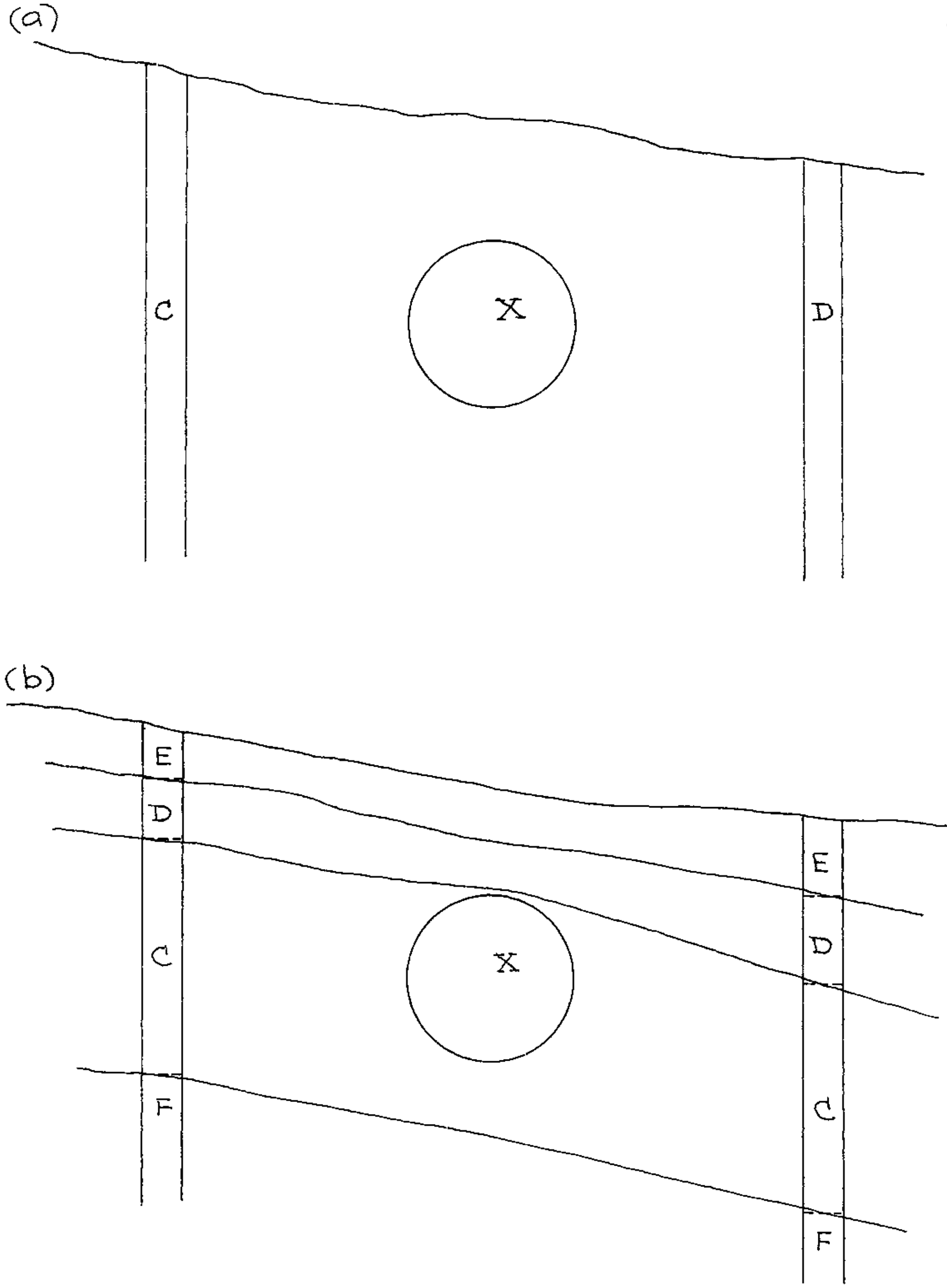


Figure 1.5 ILLUSTRATION OF ESTIMATING SOIL PROFILE UNCERTAINTIES

engineer's mind, is the probable distribution of states C, D, E, and F of α. If the layering of material properties in the ground were indeed deterministic then it could be said from this profile that the probability of state C of α occurring at point X is unity while the probability of α being D, E, or F is nil, or at X: $P[\alpha(C)] = 1$, $P[\alpha(D)] = P[\alpha(E)] = P[\alpha(F)] = 0$.

Realizing the variability of stratification of ground conditions in nature, and considering his experience with this site specifically, the engineer, deciding that the actual boundary may intersect or even dip below the tunnel, predicts with as much confidence as he can from the information available that at X: $P[\alpha(C)] = 0.7$, $P[\alpha(D)] = 0.3$, $P[\alpha(E)] = P[\alpha(F)] = 0$. If more detailed information is needed for the design, then the engineer must assess what the effect of further geologic evidence on his definition of uncertainties will be. This can be accomplished by defining the uncertainties in the testing technique and applying Bayes' Theorem, which is described below.

If the engineer has experience with a given method of site exploration in the geologic environment in which he is interested, then he may note that for every j times sample c has appeared, it has been indicative of state C k times, and of state D h times. So the probability that c will be the test result when the state of nature is either C or D is:

$$P[c/\alpha(C)] = k/j \qquad P[c/\alpha(D)] = h/j \tag{1.5a}$$

A similar relationship can be developed if the sample turns out to be d, indicative of state D or C:

$$P[d/\alpha(D)] = k_1/j_1 \qquad P[d/\alpha(C)] = h_1/j_1 \tag{1.5b}$$

where j_1, k_1, and h_1 are the results of experience and obtained in the same manner as j,k, and h. These probabilities assume that the test is an indicator of the actual state of nature and that the reliability of the tests can be accurately predicted by Equations 1.5.

The translation of test results into modifications of the previously assigned uncertainties is accomplished through Bayes' Theorem which states that the probability of a particular condition, say $\alpha(C)$, occurring when the test result is c, or $P[\alpha(C)/c]$ is equal to the intersection of the probability that the appearance of sample c will predict state C, when state C does occur (or the reliability Equation 1.5a) with the previously computed probability of the occurrence of $\alpha(C)$; divided by the sum of the intersection of all possible states which can be predicted by the appearance of result c and the previously computed probabilities of the occurrence of all these other states. In the terms of the problem presented here, Bayes' Theorem would be:

$$P[\alpha(C)/c] = \frac{P[c/\alpha(C)] \cdot P[\alpha(C)]}{\sum_{i=A}^{N} P[c/\alpha(i)] \cdot P[\alpha(i)]} = \tag{1.6}$$

$$\frac{P[c/\alpha(C)] \cdot P[\alpha(C)]}{P[c/\alpha(C)] \cdot P[\alpha(C)] + P[c/\alpha(D)] \cdot P[\alpha(D)]} =$$

$$\frac{(k/j) \cdot (0.7)}{(k/j) \cdot (0.7) + (h/j) \cdot (0.3)} = \frac{0.7(k/j)}{A}$$

where N is the set of mutually exclusive and exhaustive states of α. $P[\alpha(C)/c]$ is now the revised probability of the occurrence of end branch node $\alpha(C)$ given the new evidence, as interpreted using Bayes' Theorem. A similar expression for $P[\alpha(C)/d]$ can be found:

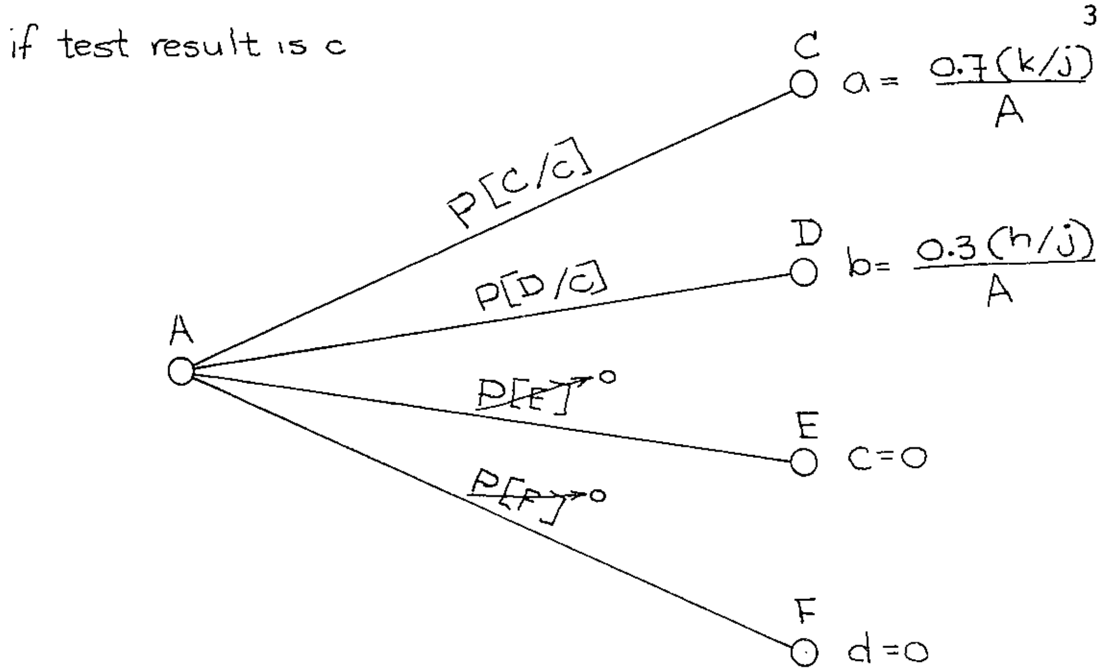
$$P[\alpha(C)/d] = \frac{(h_1/j_1) \cdot (0.7)}{(k_1/j_1) \cdot (0.3) + (h_1/j_1) \cdot (0.7)} = \frac{0.7(h_1/j_1)}{B}$$

Using Bayes' Theorem and given hypothetical test results c or d, Figure 1.2 can be reevaluated as shown in Figure 1.6. If detail is still not sufficient with respect to geology then another investigation method may be tried, again applying Bayes' Theorem.

Examples of the application of Bayes' Theorem to more complex site investigation data and further discussions of the assignment of risks are presented by Vick (39.) This introduction to Probability Theory has been meant only to present the framework within which the geologic model will be structured.

1.4 Evaluation of Tunnel/Ground Interaction

Once the tunnel cost model, which is sensitive to



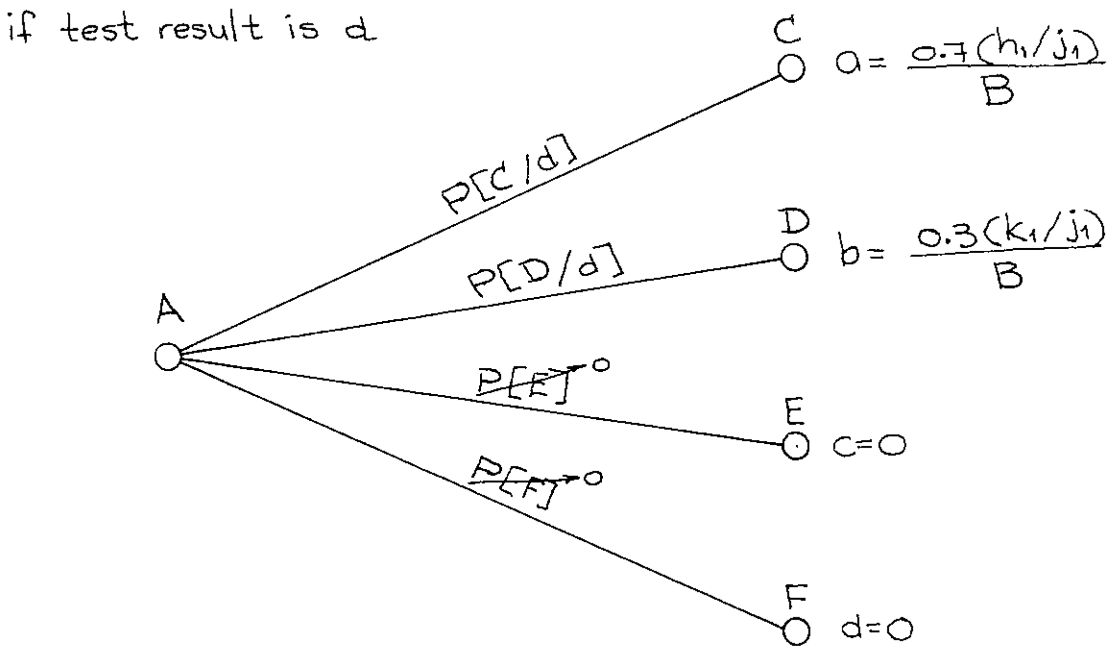


Figure 1.6 ADJUSTMENT OF PROBABILITIES ACCORDING TO SITE INVESTIGATION OUTCOME c OR d

changes in ground conditions, has been formulated and the probabilistic interpretation of geology has been structured into a complementary decision tree format; it is necessary to somehow interact these two models so that the engineer can use the results as a basis for decision-making. A criterion for this evaluation, similar to the one employed by Vick (39) with respect to hard rock tunneling (and the one which will be employed here as an example,) is the computation of equivalent monetary values (EMV) for a decision tree with respect to one particular tunnel design.

The decision tree is structured using the same ground parameters as the tunnel cost model, such that each possible ground condition for one parameter (i.e. for all the branches stemming from the same node) must have a cost consequence, κ, which can be obtained from analysis of each ground condition using the cost model. Figure 1.7 illustrates the calculation of end branch node equivalent monetary values (bemv) for the α parameter states of Figure 1.2 which can be expressed mathematically as:

$$\mathrm{bemv}(n) = n\kappa_n = \kappa_n \cdot P[\alpha(N)] \qquad [1.7]$$

where N is one of the mutually exclusive, exhaustive states of the parameter α corresponding to end branch node probability, n, κ_n is the cost consequence of the n^{th} node computed from the cost model, and bemv(n) is the branch equivalent monetary value of the n^{th} branch node stemming from

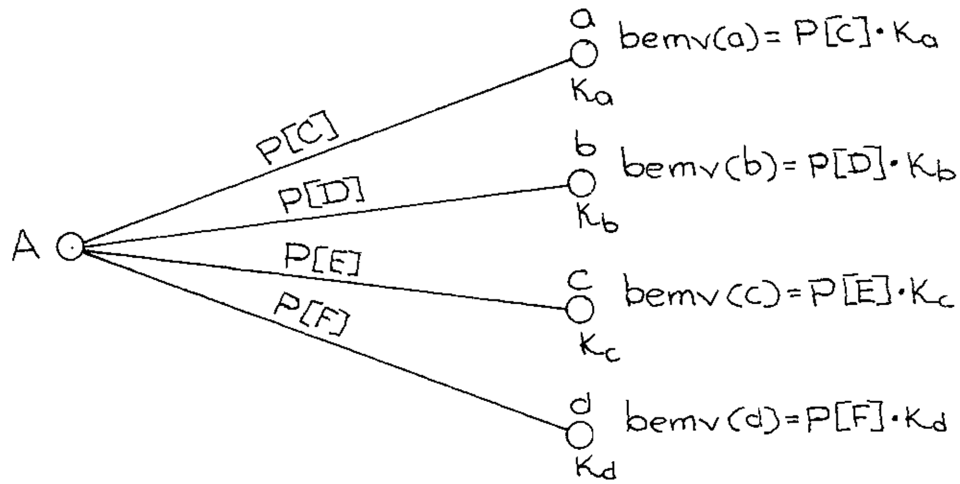


Figure 1.7 COMPUTATION OF END BRANCH NODE MONETARY VALUES

node A. The summation, over all the branches from node A, of bemv results in the computation of the equivalent monetary value (emv) for the α parameter states with cost consequences κ for each end branch node, or:

$$emv(A) = P[\alpha(C)] \cdot \kappa_a + P[\alpha(D)] \cdot$$

$$\kappa_b + P[\alpha(E)] \cdot \kappa_c + P[\alpha(F)] \cdot \kappa_d =$$

$$a\kappa_a + b\kappa_b + c\kappa_c + d\kappa_d =$$

$$\sum_{n=a}^{d} n\kappa_n \qquad [1.8]$$

If the decision tree structure of Figure 1.7 was now used with a different design which, when modeled with respect to the α parameter states, resulted in cost consequences of π_n for the N parameter states, then an emv for the α parameter states from node A could be computed in a fashion identical to the way Equation 1.7 was calculated, with the result:

$$emv(A)_\pi = \sum_{n=a}^{d} n\pi_n \qquad [1.9]$$

If the equivalent monetary criterion is accepted by the engineer as a valid means of evaluating the relative merit of two designs; which have respective cost consequences κ_n and π_n when, for each of N variations of the state of geologic parameter α, the design encounters the probabilistic interpretation of geology represented by Figure 1.2; then a comparison of Equations 1.9 and 1.8 will reveal which of the tunnel designs is more economic for the assumed ground conditions. The inequality: $emv(A)_\pi < emv(A)_\kappa$ will indicate that the design with cost consequences π_n will be more economical than the design with cost consequences κ_n; while: $emv(A)_\pi > emv(A)_\kappa$ indicates the opposite with respect to the designs and ground parameter α.

It must be remembered that emv is only the summation of values over one set of branches which emanate from one node. It is necessary to evaluate the design over the entire set of ground parameters which define the geology along the tunnel alignment. This can be accomplished if it can be recalled that for a decision tree representing more than one set of ground parameters, (as was illustrated in Figure 1.4) the end branch node probabilities, a through h, can be computed as the product of all the branch probabilities between a given end branch node and the starting node, A. The equivalent monetary value for the entire decision tree (EMV) can be computed from the summation of the products of the end branch node probabilities and the total cost consequences evaluated along the paths from node A to the end branch nodes.

To illustrate the above, consider the single branch of the decision tree of Figure 1.4 which contains nodes A, C, and G. Each node has an associated ground condition with a definite cost consequence, κ, which affects the total cost of the tunnel. In other words, at node A the total cost of the tunnel is T; at node C, because ground state C of parameter α is encountered, the total cost is changed by the cost consequence of node C, κ_1; and at node G the total is changed by the cost consequence associated with state G of parameter β, κ_2. At the end branch node where the branch probability is a, the new tunnel construction cost which is the result of the occurrence of A, C, and G, K_a, is equal to the total cost of A, T_A, plus the cost consequences of C and G, κ_1 and κ_2 respectively, or: $K_a = T_A + \kappa_1 + \kappa_2$. A more general expression which would include the cost consequences κ_1 to κ_{m-1} of a branch with m nodes and end branch node probability, n, would be:

$$K_n = T + \sum_{i=1}^{m-1} \kappa_i \qquad [1.10]$$

Then to compute EMV over the entire decision tree it is necessary to sum the product of the end branch cost consequences, K_n, with their respective end branch node probabilities, n, of j total branches:

$$EMV = \sum_{j=a}^{n} jK_j \qquad [1.11]$$

The following example illustrates the application of the equivalent monetary value criterion on Bayes' Theorem to a purely hypothetical tunnel problem.

Suppose that the ground parameter of interest, α, defined as the organic material parameter with states C or D (which are the presence and absence of organic material respectively) has originally been considered to be D, absence, at some point X above the proposed tunnel crown. A new borehole at location R has disclosed the presence of organic material, $\alpha(C)$, at the elevation of point X and the engineer is confronted with the situation illustrated in Figure 1.8. He is concerned with the possibility of $\alpha(C)$ occurring at X, endangering the building structure, and estimates that if there were any organic material at X the cost difference , κ, would be \$30,000, which would quadruple the cost of the short segment of tunnel with the particular design under consideration. With the evidence from borehole R he decides, considering the geology of the area that there is indeed a 20% chance that $\alpha(C)$ exists at X, and structures a decision tree as in Figure 1.9, with the two end branch probabilities

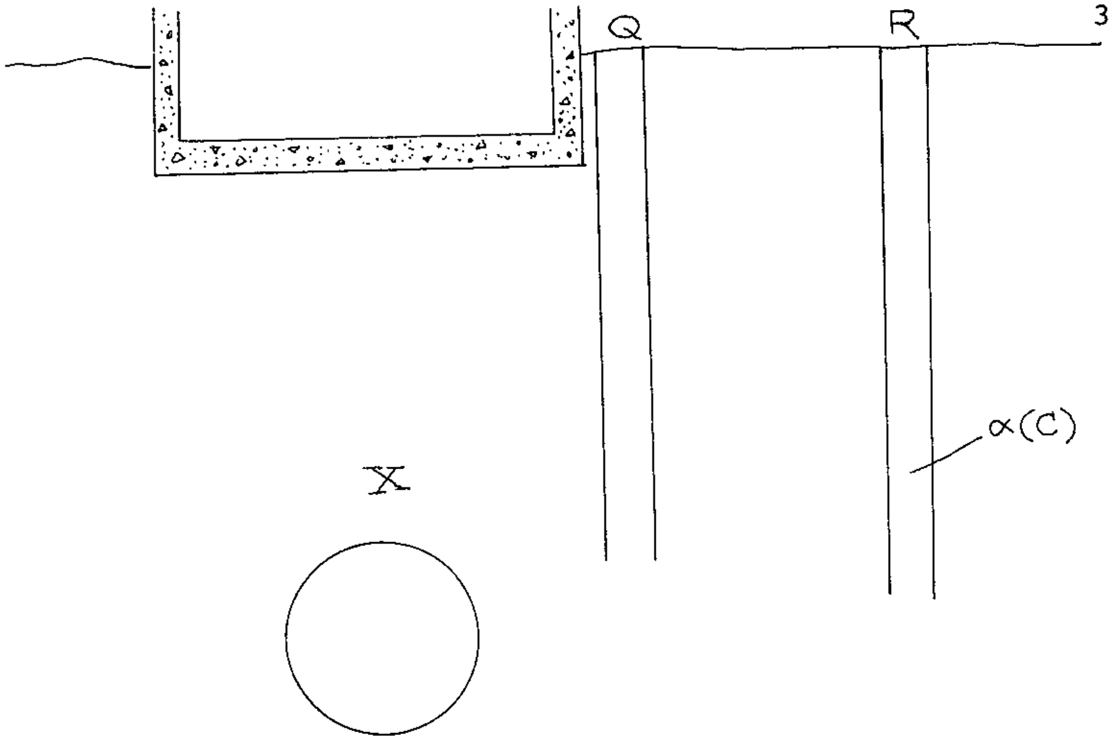


Figure 1.8 EVALUATION OF UNCERTAINTY AT POINT X

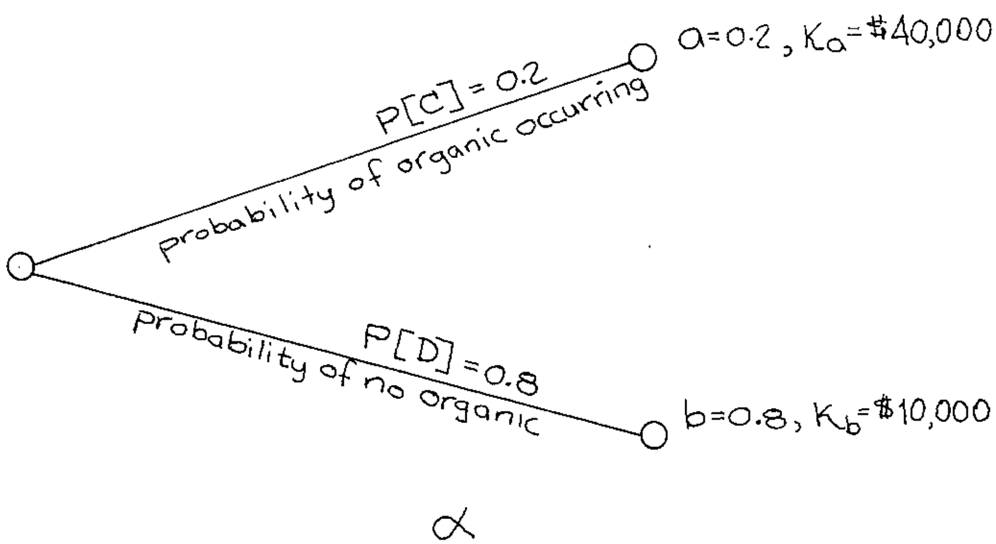


Figure 1.9 EMV COMPUTATION EXAMPLE

a and b, and cost consequences, κ_1 and κ_2, being:

$$a = P[\alpha(C)] = 0.2, \text{ and } \kappa_a = \$40,000$$

$$b = P[\alpha(D)] = 0.8, \text{ and } \kappa_b = \$10,000$$

The EMV for the tree is (evaluating Equation 1.11 for n = a and b:)

$$EMV = a\kappa_a + b\kappa_b$$

or:

$$EMV = 0.2 \cdot \$40,000 + 0.8 \cdot \$10,000 = \$16,000$$

which in his opinion may indicate a significant difference in terms of his design evaluation. He could then try another borehole at, say, Q which he could use as additional evidence of the occurrence of α(C.) Deciding whether or not to continue with further investigation, he decides that if point X has state C then the chance that an organic sample, A_o, will be obtained from a borehole at Q is only 50%, while if there is no organic present, an inorganic sample, A_1, is 90% certain, based on his evaluation of the deficiencies of borehole sampling and the relation of borehole Q to point X. Expressing these relations in terms of conditional probabilities:

$$P[A_o/\alpha(C)] = .5 \qquad P[A_1/\alpha(C)] = .5$$

$$P[A_o/\alpha(D)] = .1 \qquad P[A_1/\alpha(D)] = .9$$

Applying Baye's Theorem, Equation 1.6:

$$P[\alpha(C)/A_o] = \frac{P[A_o/\alpha(C)]P[\alpha(C)]}{P[A_o/\alpha(C)]P[\alpha(C)] + P[A_o/\alpha(D)]P[\alpha(D)]}$$

$$= \frac{0.5 \cdot 0.2}{0.5 \cdot 0.2 + 0.1 \cdot 0.8} = 0.56$$

$$P[\alpha(D)/A_0] = \frac{0.1 \cdot 0.8}{0.5 \cdot 0.2 + 0.1 \cdot 0.8} = 0.44$$

$$P[\alpha(C)/A_1] = \frac{0.5 \cdot 0.2}{0.5 \cdot 0.2 + 0.9 \cdot 0.8} = 0.12$$

$$P[\alpha(D)/A_1] = \frac{0.9 \cdot 0.8}{0.5 \cdot 0.2 + 0.9 \cdot 0.8} = 0.88$$

If the result were A_0 (an organic sample) his reassessment would lead to new end branch node probabilities for Figure 1.9 of: $a_0 = P[\alpha(C)/A_0] = 0.56$

$b_0 = P[\alpha(D)/A_0] = 0.44$

and the EMV, if sample A_0 occurs, will be: $EMV_0 = a\kappa_a + b\kappa_b = 0.56(\$40,000) + 0.44(\$10,000) = \$26,800$. If the sample from the borehole disclosed A_1 (an inorganic) then:

$a_1 = P[\alpha(C)/A_1] = 0.12$

$b_1 = P[\alpha(D)/A_1] = 0.88$

and $EMV_1 = 0.12(\$40,000) + 0.88(\$10,000) = \$13,600$.

Comparing the three EMV's:

$EMV = \$16,000$ (original)

$EMV_0 = \$26,800$

$EMV_1 = \$13,600$

The engineer decides that if an additional sample at Q turned out to be A_0, it would have a considerable impact on his design, while a sample A_1 would be more complex to evaluate. He therefore samples, obtains a result A_1 and for the present, decides to continue with the analysis reserving final judgement of whether more exploration is necessary to determine α at X until a time when he can put it into better perspective with the total cost of the project.

He next turns his attention to the occurrence of gas, parameter β, beneath the building and over the tunnel. The gas could be from either of two sources, organic matter, if it exists, or from leaky utility lines. Now the designer must evaluate the possibility of the occurrence of gas conditionally, dependent on the occurrence of organic matter. The designer decides where:

$\alpha(C) \equiv$ organic matter $\quad\beta(G) \equiv$ gas

$\alpha(D) \equiv$ no organic matter $\beta(H) \equiv$ no gas

$P[\beta(G)/\alpha(C)] = 0.9 \qquad P[\beta(G)/\alpha(D)] = 0.1$

$P[\beta(H)/\alpha(C)] = 0.1 \qquad P[\beta(H)/\alpha(D)] = 0.9$

and he adds to the decision tree of Figure 1.9 to arrive at a new decision tree with new end branch cost consequences (see Figure 1.10.) To evaluate the EMV for this tree, he uses Equation 1.11 where n = a, b, c, and d:

$$EMV = aK_a + bK_b + cK_c + dK_d =$$

$$P[C]P[G/C](\$45,000) + P[C]P[H/C](\$40,000) +$$

$$P[D]P[G/D](\$15,000) + P[D]P[H/D](\$10,000) =$$

$$0.12 \cdot 0.9(\$45,000) + 0.12 \cdot 0.1(\$40,000) +$$

$$0.88 \cdot 0.1(\$15,000) + 0.88 \cdot 0.9(\$10,000) =$$

$$\$14,580$$

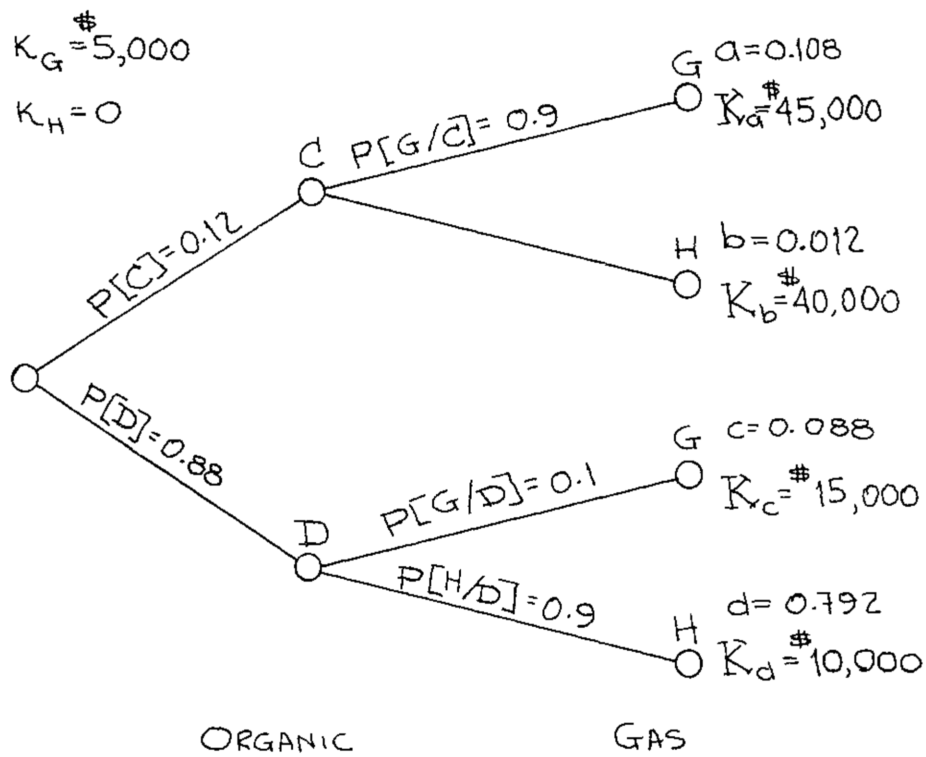


Figure 1.10 ILLUSTRATION OF APPLICATION OF EMV
TO DECISION TREES

It must be remembered that the above is only an example of the design approach only, simply meant to acquaint the reader with the basic principles involved. The steps which must be taken before this approach can be applied to the design of a real tunnel is first the identification of those parameters which, when combined, will adequately describe the ground conditions for tunnel design; and secondly further development of the tunnel model to see exactly how changes in ground conditions can be translated into costs.

1.5 OUTLINE OF REMAINDER OF THESIS

In this chapter, a general approach to tunnel system design has been proposed which incorporates probability and decision theory to help evaluate the effect of the uncertainty of geologic conditions on the tunneling process. The introduction has seemed, no doubt, somewhat long. Many new tools have been exposed however, tools which are not in general familiar to the tunnel designer; and a detailed introduction to these therefore necessitated this great length.

The remainder of this work will be devoted to developing these tools so that they can specifically be applied to soft ground tunneling in the urban environment. This task requires that both the ground conditions which affect tunneling, and the components of the tunnel construction process itself, be reviewed and analyzed, the better to represent them with the models which are being developed. To show how this whole topic will be approached, since it requires a momentary digression from the line of thought begun in the introduction; an outline of the ground to be covered follows.

Chapter 2 will attempt to provide the basis for formulating the specific geologic model for the soft ground tunneling process. It will begin by reviewing soil/structure interaction which is the basis for tunnel structural design, and then mate the soil/structure interaction with ground/construction interaction which is the basis for construction planning. These two topics are developed in this manner because the approach being proposed considers both steps to be part of the tunnel design process. Then, the geologic parameters which affect the combined ground/structure/construction are reviewed and a simple ground classification for the purpose of the tunnel model is proposed.

Chapter 3 reviews common tunnel construction systems with the purpose of identifying how they are affected by changes in ground conditions. Various relations reflecting the adequacy of a given construction system to a particular ground condition are discussed.

The approach is synthesized in Chapter 4. A more detailed description of the tunnel cost model is presented, and decision trees for the modeling of soft ground urban tunnels are presented. The application of these models to tunneling is discussed.

GROUND CONDITIONS

2.1 INTRODUCTION

A decision tree framework has been proposed as a probabilistic model of geology for tunnel design. The problem now is one of identifying those ground parameters which, when modeled and translated into cost consequences; will adequately describe the ground dependent component of tunnel cost. These ground parameters must be predictable within a relatively identifiable range of uncertainty and, of course, must have a cost consequence when analyzed with the tunnel cost model. Identification of major ground variables in tunneling has been studied by Schmidt et al (32,) Ash et al (2,) and Heuer (18.) Schmidt (31) and Ash et al (2) have proposed schemes of systematic site investigation meant to minimize geologic uncertainty.

The interaction of the tunneling process with the ground appears to be identifiable on two different planes, a subtle and a gross level. The subtle level is the influence of ground variation on the interaction of soil and structure which is really a design problem. The load on the tunnel liner, for instance, or the length of time an excavated cavity can stand unsupported (standup time,) are functions of the engineering properties of the ground as a construction material, i.e. the material properties. The gross level of ground effect can be referred to as ground/

construction interaction and is the way in which the geologic properties of the ground, such as strata discontinuities or the water table elevation, influence construction processes such as the rate of excavation or amount (in volume per unit time) of dewatering effort. A representative model of the ground will include geologic parameters which influence both soil/structure and ground/construction aspects of the tunneling process.

This chapter will examine in some detail the nature of soil/structure interaction and the material properties which influence it. Ground/construction interaction will also be reviewed from the point of view of identifying those geologic parameters which can have measurable effect.

2.2 Soil/Structure Interaction

When a circular cavity is opened in a soil mass, there is an immediate release of stresses around the circumference of the excavation. If there were no change in stresses at any time during the construction of the tunnel, or as Deere et al (10) put it, one could "wish a circular tunnel liner into existence;" and if the support were perfectly rigid; then the stresses on the tunnel would be as shown in Figure 2.1. If the tunnel were perfectly flexible, that is if there were no lining or support present, then the circular cavity would deform until an equilibrium of stresses around the

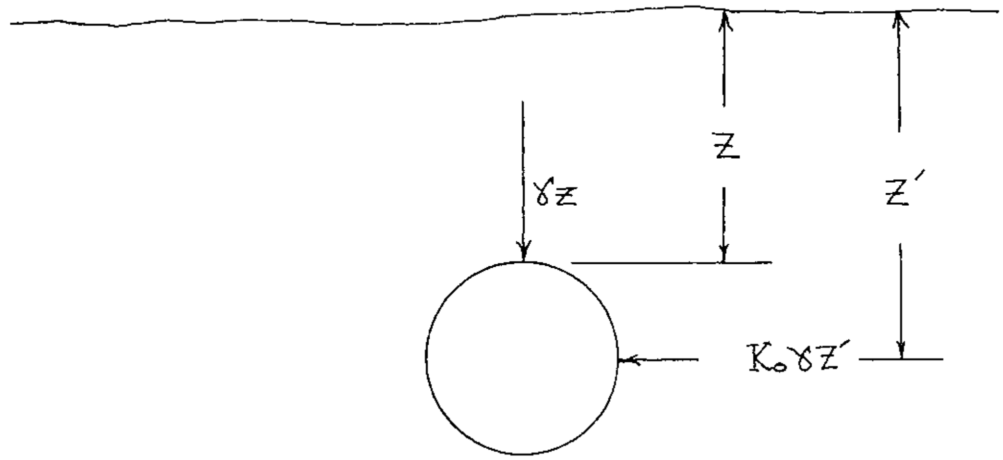


Figure 2.1 STRESSES ON A RIGID CIRCULAR LINING "WISHED" INTO PLACE IN A SOIL MASS

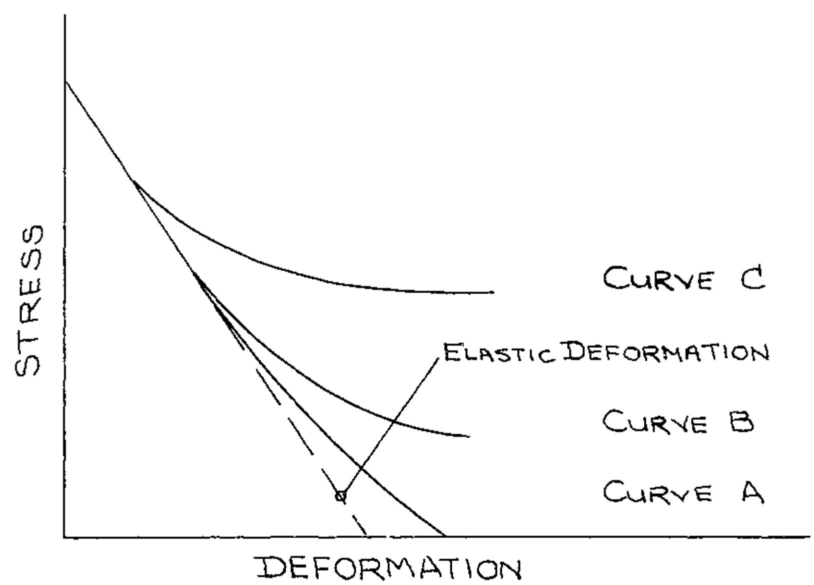


Figure 2.2 STRESS-DEFORMATION CURVES FOR A POINT ON THE CROWN OF A CIRCULAR OPENING IN A SOIL MASS

cavity was achieved. In either the flexible or rigid case, the stresses can reach a maximum equal to the shear strength of the soil. Once the stresses reach the shear strength, plastic deformation occurs, transferring the stresses into the soil mass until all loads are, in effect, resisted by the soil.

In an unsupported cavity therefore, the deformation at the outset is linear as shown by the dashed line in Figure 2.2. Since soil is not a linear elastic material through all stress ranges, at some point the deformation may become inelastic and continue so until all stress is relieved, as in Curve A, and the excavation becomes stable. If equilibrium is never achieved, as in Curves B and C, the cavity collapses, a sure sign of failure.

In actual tunnel construction the cavity is without support for some time between initial excavation and the erection of the liner. This process is wholly dependent on construction techniques, although these can be influenced by such ground parameters as soil/water conditions or ground strength. The time to erect one liner ring in normal conditions is still a function of the performance of the tunnel working crew and the design of the liner.

If one defines success in tunnel construction in terms of soil/structure interaction alone, success or failure can be expressed by means of such stress-deformation curves as

Figure 2.3 which show the interaction between ground and liner. If success is defined as the application of the tunnel support prior to complete collapse of the cavity then the construction of a tunnel with support characteristics as shown by the solid line representing liner deflection in Figure 2.3, in grounds represented by curves A and B, will be a success; while it will be a failure in ground C because collapse had occurred before support. If the definition of success, however, were to include as a criterion a maximum allowable deformation of ground, represented by point X_1 on the deformation axis; then, as shown, tunneling in ground B also becomes a failure because of excess deformation. If X_2 represented the maximum allowable deformation, then all efforts would be a failure and either the construction procedure would have to change (as in the dashed line of support bahavior) or the ground behavior would have to be modified to allow construction.

The success or failure (stability) of a tunnel constructed by a particular procedure can be predicted if the time/deformation behavior of the ground is known. The time from initial excavation to failure (by whatever standard of failure is apropos for the particular tunnel in question,) is a reasonable value for the ground standup time, t_s. If X_1 were the failure criterion then the time that it takes for a point at the crown in ground C of Figure 2.3 to displace to

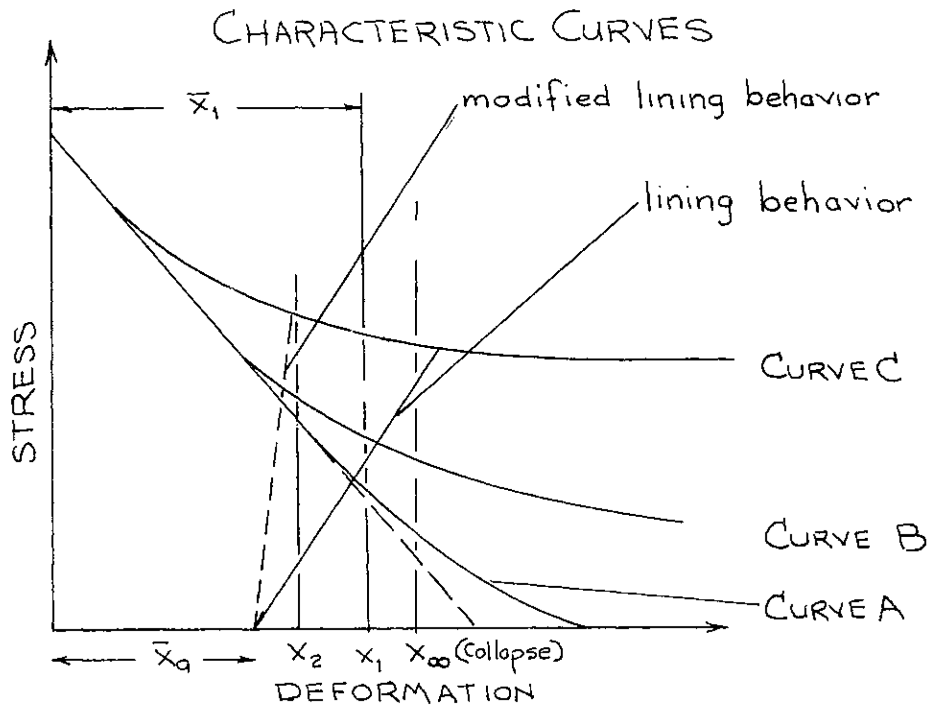


Figure 2.3 SOIL-STRUCTURE INTERACTION FOR LINER AND THREE HYPOTHETICAL GROUND CONDITIONS

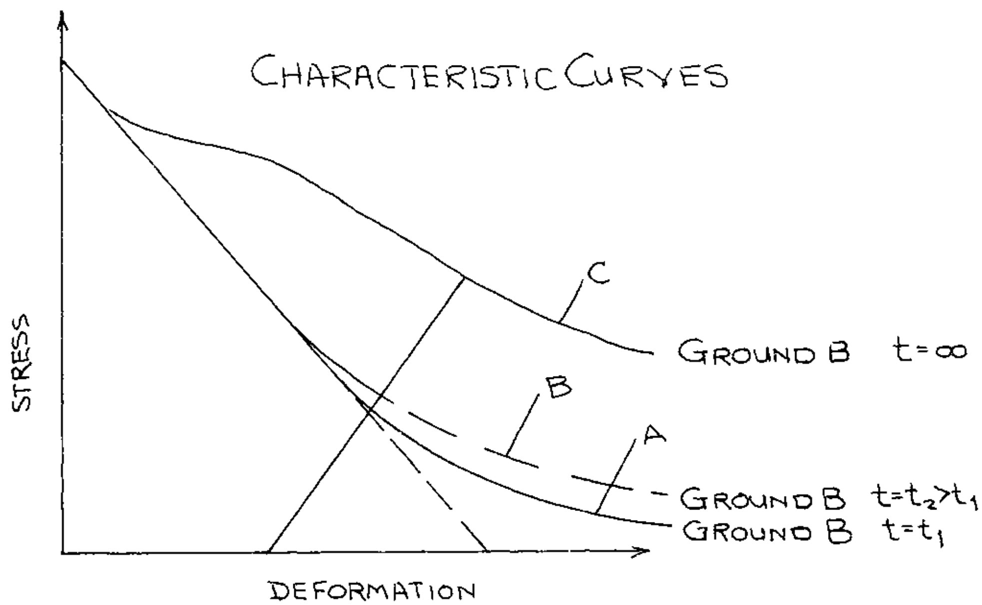


Figure 2.4 TIME DEPENDENT BEHAVIOR OF A TUNNEL IN A SWELLING SOIL

point c, or the distance $\bar{X}_1$, is the standup time t_s. If complete collapse was the criterion, then the standup time is equal to the time it takes a deformation of $\bar{X}_\infty$ to occur, as shown in the figure. The stability of the tunnel can then be forecast by a knowledge of the time necessary to erect the support, t_a, which is the time it takes for the ground to displace a distance $\bar{X}_a$. Therefore, the comparison of t_s to t_a is actually a comparison of the deformation in the ground allowed by the construction system compared to the total allowable deformation before failure occurs. It is possible then to derive an index of the relative stability of the ground compared to the construction procedure employed. This index will be called the standup time ratio, STR, and be equal to the ratio of the ground standup time to the support erection time, or:

$$STR = t_s/t_a \qquad [2.1]$$

If STR >>1 then the ground will be stable during construction, while an STR <<1 predicts instability or the impracticability of a certain construction technique in a particular ground condition. Where STR ≈ 1, the stiffness of the support system becomes important and must be analyzed. Such an analysis, in the region of STR = 1, must include decisions regarding possible variations in t_s and t_a. Caution must be exercised because even though t_s is chiefly a function of ground properties and t_a of construction parameters, there is a dependence of each term on the other. The absolute stability of the ground may be affected by the interference of the construction with soil parameters, e.g. cohesion, which are important components of the stability. The action of the ground conditions on t_a are discussed further below.

In homogeneous material, variation in normal ground behavior contributes greatly to the uncertainty of design analyses because of the poorly understood mechanisms which govern some material behavior with time. In plastic soils, for instance, sudden unloading will cause negative pore pressures within the soil, tending to draw more water into the soil. The result is that the behavior of the ground may be as in curve A in Figure 2.4 at the time of support application, t_1; but at some time thereafter, t_2, after the pore pressure has had a chance to stabilize, the stress on the support may increase as the ground behavior changes to that of curve B. For systems where the factor of safety of supports against long term failure is small, this may be significant.

In some types of ground the behavior at time of support may be again as represented by curve A, but this time the release of stresses may trigger a swelling mechanism and the ground behavior may alter radically to that of curve C. This reaction may take place over a long period of time and put into jeopardy the final structure.

Changes in ground behavior after support installation can also be caused by such things as the cessation of temporary ground supports, e.g. end of dewatering or release of compressed air; or by enviromnental changes in the vicinity of the structure as with the driving of a parallel tunnel. So the ground behavior must also be considered a function of the construction method, and these considerations included in the analyses.

Another problem altogether is the variation of the characteristic curve of ground behavior which can occur at any time. This curve is a function of the elastic and plastic properties of the ground and is controlled by such ground characteristics as permeability and shear strength, and is also time dependent. The designer must be very aware of the limitations of measuring these properties, and his degree of confidence in the data from which these curves are drawn also will affect the shape of the curves. These are the uncertainties controlled by error in site investigation methods or deviations in material behavior away from the sample that was tested.

2.3 Ground Parameters Influencing Soil/Structure Interaction

The intrinsic engineering properties of the ground material being tunneled are most important with respect to the stability of the tunnel. The properties of each component material of each individual layer determines the behavior of the mass. If there are n layers of materials with varying values of some material parameter α, and ∇ is the mass parameter which is the manifestation of the contribution of the α parameter of each of the n layers; then the contribution of each individual layer can be said to be some factor ζ, where the sum of ζ for all layers is unity, or:

$$\sum_{1}^{n} \zeta_n = 1 \qquad [2.2]$$

then:

$$\nabla = \sum_{1}^{n} \zeta_n \cdot \alpha_n \qquad [2.3]$$

The problem in assessing the mass parameter ∇, for any segment where ∇ is assumed to remain constant, is twofold. First each layer of material with a value of α different from the layer above it and/or below it must be identified. This assumes that there is a measurable and significant change in α commencing at the boundary between layers. Worse, this assumes that the boundary can be located within a tolerance of less than the thickness of the layer itself. The complexity of determining α increases as the number of layers, n, increases with respect to the tunnel cross-section (the thickness of layers decreasing.) As the number of layers becomes very large ($n \to \infty$) the behavior of the ground approaches a homogeneity which can be evaluated in the same manner as the one layer system was. Therefore the ground parameters referred to will be meant to be those which are of importance in a given ground configuration. The two different parameters which will be used will be material parameters for systems where the number of layers is either very small or very large; and mass parameters where either discontinuities or boundaries are of most importance.

2.3.1 Index Properties

An index property is a value which has been obtained from soil samples by standardized measurements and has some meaning in relation to the behavior of that one soil type under some given conditions. This meaning is inferred by the engineer based on the results of many of these standardized measurements over a wide range of soils. Such properties are useful in categorizing materials into groups which have similar behavioristic properties under given loading conditions. Many useful correlations of index properties (and further discussion of the use of index properties) can be found in Lambe and Whitman (21,) Peck et al (26,) and DM-7(11.) Some important index properties in relation to tunneling are listed below. They are generally divided into two all encompassing categories, cohesive and cohesionless soils. The difference between the two categories is that the soil grains in the cohesionless category are large enough to be controlled by gravitational forces, rather than the forces between particles (body forces.) The behavior of cohesive on the other hand is influenced more by the interparticle forces than the gravitational. In nature, however, it is sometimes difficult to distinguish to what extent a material is controlled by either.

Cohesionless

a. Grain Size Distribution Curve

b. Effective Grain Size, D_{10}

c. Uniformity Coefficient, U

d. Relative Density, D_r

e. Internal Angle of Friction, ϕ

Cohesive

a. Atterberg Limits

b. Consolidation Characteristics

c. Strength Parameters

2.3.2 Soil Strength Properties

The stability of an opening of defined geometry, at a given depth in a reasonably known geology, tunneled by a particular construction procedure; is mainly a function of the soil strength properties, especially as related to time and displacement. The loads that a support will be subject to, both long term and immediate, are also a function of these properties. The behavior of the ground in this respect can be classified in several ways depending upon the material properties of the ground; and again this behavior can be categorized as either granular or cohesive, depending upon whether the drained or undrained strengths govern.

The excavation of a vertical face in a clean, dry, granular material with no cementation is impossible. Some amount of confinement must be provided or the soil will run until the slope of the excavation is equal to the angle of repose, ϕ_r, of the material (≈30°.) Below the ground water table, a seepage pressure will exist at the face, which could cause the material to flow into the excavation. The most serious consequence, depending on the seepage pressure, D_{10}, and D_r; would be the liquefaction of the material at the face and a flow of the ground into the excavation which could fill the tunnel. The dividing line for this type of behavior is not arbitrarily particle size, as say the division between sand and silt by the Unified Classification System; but is also dependent on the shape of the particle and the cementation if any between particles. A small amount of cementation can, in fact, increase the critical gradient necessary to initiate flow, and may even delay running of the soil for some finite length of time (31.)

Many vertical excavations in granular soils will stand for periods of time because of either true or apparent cohesion. Cementation of granular particles is possible especially with the presence of a cementing agent. Small amounts of cohesive soils filling the voids between particles can

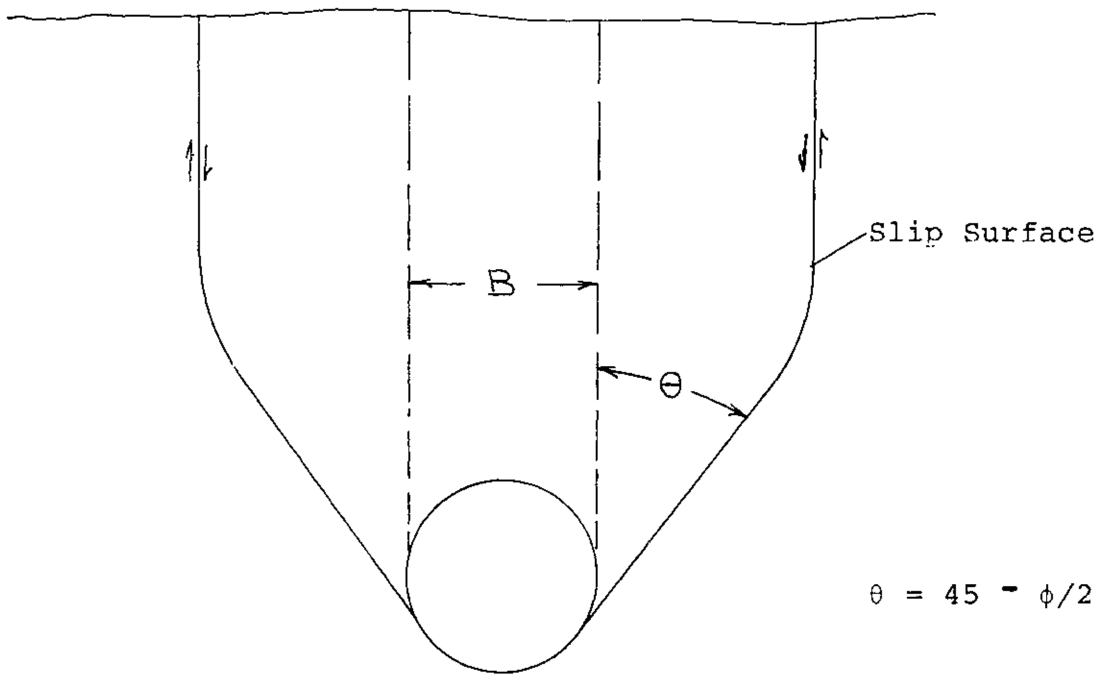


Figure 2.5 ARCHING ABOVE A CIRCULAR EXCAVATION

produce the same effect through negative pore water pressures or capillary tension can act similarly in clean small grained granular soils. These latter two mechanisms however act only for short periods of time as the water drains. Measurement of apparent cohesion is impossible as is the absolute prediction of the occurrence of true cementation; but utilizing model tests and experience with the local geology, or with similar soils, these properties may be predicted for use in design.

Stability in granular soils is far more amenable to theory, and much work has been performed especially in developing arching theories (36)(10.) When an opening is excavated in a cohesionless medium, a wedge of material begins to slide into the opening as in Figure 2.5. As the material deforms, shearing resistance develops within the soil to resist the deformation until enough develops so that movement stops. The shearing strength of the material is a function of the relative density of the material. Here the differing properties of a loose and dense medium when subjected to shear becomes very important as the dense material tends to expand and the loose one to decrease in volume.

In a cohesive material the very low permeability does not allow a rapid equalization of pore pressures and consequently for the unsupported lifetime of a cohesive material the behavior is dependent upon the undrained properties. The stability of the face and the walls is controlled by the ratio of the overburden pressure to the undrained shear strength. Peck (25) cited the criterion of Broms and Bennermark for the stability of plastic soils at depths greater than two diameters; a stability ratio, SR, could be defined as:

$$SR = (p_z - p_i)/S_u < 6 \qquad [2.4(a)]$$

Where:

$$p_z = \sum_{1}^{n} \gamma_n H_n \qquad [2.4(b)]$$

$p_z \equiv$ total vertical stress at depth z

$p_i \equiv$ internal pressure on tunnel (air, slurry, etc.)

$n \equiv$ number of layers soil from surface to depth z

$\gamma_n \equiv$ total unit weight of material in layer n

$H_n \equiv$ height of layer n

The conclusion that Peck drew from the case histories evaluated in his paper was that the actual stability ratio was closer to 5.

In evaluating the feasibility of a design, SR >6 indicates that some support is necessary, while SR <1 indicates an elastic, stable behavior is to be expected. Figure 2.6 summarizes some of the behavior that can be deduced from a knowledge of the undrained shear strength.

Another important aspect of undrained behavior, for ϕ assumed equal to 0, is the plastic zone which develops as the stresses reach s_u around the excavation. The width of this zone, for $K_o = 1$ and assuming the Mohr-Coulomb failure criterion where:

$$(\sigma_1 - \sigma_3)/2 = s_u \qquad [2.5(a)]$$

is then the radius of the plastic zone, R, according to Deere et al (10) defined as:

$$R = a \exp\{(p_z - p_i)/2s_u - (1/2)\} \qquad [2.5(b)]$$

where a $\equiv$ radius of tunnel and all else as in Equations 2.4.

The value of s_u can vary depending on the methods used to determine it, and laboratory tests of course add many variables. Perhaps the most useful guides to a designer are correlations with index properties, e.g. P.I. or w_L for nor-

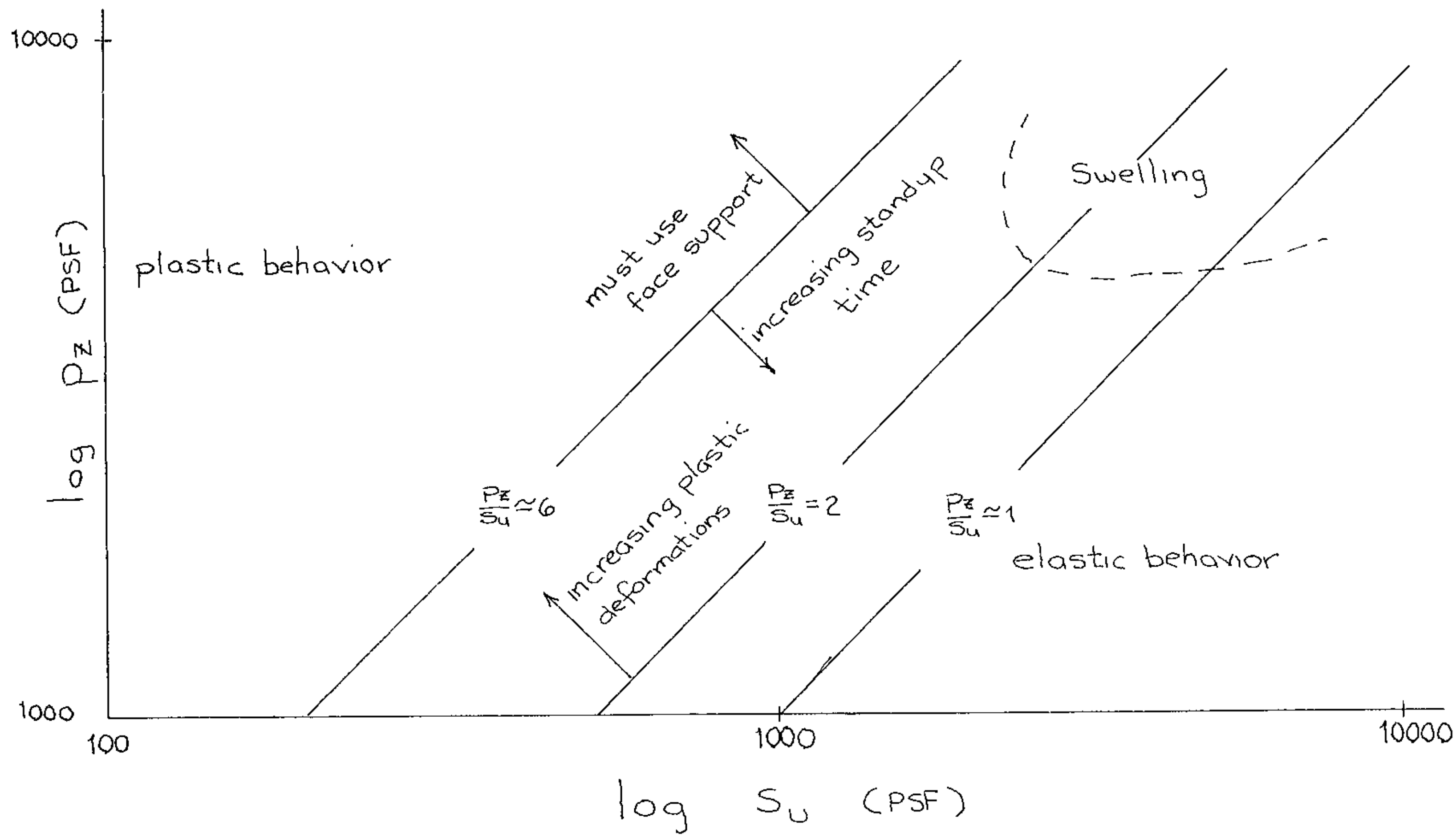


Figure 2.6 RELATION OF STABILITY RATIO TO TUNNELING PROCESS
after Schmidt et al (32)

mally consolidated clays, or relations of s_u to OCR. Often, in an area where a great deal of experience with performance values of s_u exists, there are local correlations of strength to index properties.

2.3.3 Permeability

The importance of permeability has been inferred throughout this chapter. The rate at which water leaves the pore spaces is the factor which determines whether the behavior of a material is drained or undrained. The dissipation of pore pressures in cohesive materials controls the increase total stresses and therefore the deformation of the material. In granular materials the seepage gradient through the soil is controlled by the permeability and therefore the stability of the excavation is greatly influenced by the value of the permeability.

Part of the problem of what value to use for permeability in tunnel design is that soil is very often anisotropic and inhomogeneous, especially with respect to permeability. Where there are many layers of varying permeability, the seepage gradient may be much higher than predicted from theory because the flow will tend to concentrate in the more permeable layers. Sometimes full scale pumping tests are necessary to determine dewatering requirements because results from permeability analyses are misleading.

2.4 Ground/Construction Interaction

The arrangement of soil with varying material property and the appearance of geologic discontinuities such as boulders or the soil/rock interface is of great importance to the construction process. Soil is traditionally divided into four categories, clay, silt, sand, and gravel (with subclasses of organic materials and boulders especially important to tunneling.) The tunneling processes, such as excavating or muck handling, are very much influenced by the geology, either through variations within the processes which can be measured as rate of advance or quantity; or by necessitating a change to a different construction system when a variation in material occurs. Some of the details of geology which influence construction are listed in Table 2.1.

1. Stratification of Materials of Differing Soil Properties
2. Position of Water Table
3. Sources of Underground Water
4. Soil/Rock Interface
5. Boulder Occurrence
6. Gas Occurrence

Table 2.1 GEOLOGIC DETAILS WHICH INFLUENCE TUNNEL CONSTRUCTION

The arrangement, or stratification, of the soil materials in the ground including the ground water level and the bedrock surface have been identified by Schmidt et al(32) as the most significant geologic information for tunnel construction. This is because the nature of the material encountered, whether cohesive, cohesionless, or rock (or a mixture of any of these,) has a great impact on construction costs. Whether the tunnel will be in a uniform medium or in several soil layers with boundaries which will have a great but indeterminate effect on behavior; whether the tunnel will be under water or in the dry; or whether or not the tunnel will encounter the bedrock surface, all have significant impact on construction procedures and costs. Boulders can slow construction tremendously because they are difficult and time-consuming to remove. Gas can have disastrous effects when encountered unexpectedly. These all have direct influence on construction of themselves and therefore their occurrence must be noted. Other more subtle geologic information can also lead to a better understanding of ground behavior. A knowledge of the sources of underground water can help the engineer to estimate the severity of water-related problems if they occur.

The occurrence of these features is related to the origin of the geologic deposit in question and the origins are twofold in nature. The soil could either have been transported to its position by some means or it could have been transformed into soil from its parent rock in place.

The characteristics of a geologic deposit vary considerably depending upon its mode of transport or its parent material. Some of the properties characteristic of geologic origins are listed in Table 2.2. The occurrence of those important geologic parameters listed in Table 2.1 can be somewhat predicted from a study of the geology of an area. Evidence from site investigations and previous experiences can be used to bolster the degree of confidence in the possible occurrence of a particular state.

The problem is that the formation of the geology of a particular area can be a very complex arrangement of all of the mechanisms listed in Table 2.2. The effects of glaciation in particular, because of their extremely heterogeneous nature, can be very difficult to diagnose. One may be able to look at several samples of heterogeneous alluvium and not know whether to expect the presence of boulders or not, for example. If the deposits could be traced to glacio-fluvial activity then there might be a possibility of boulders, while if these were ordinary river deposits the chance of boulder occurrence would be much less. Fortunately the geology of most heavily populated areas has been quite extensively researched and much useful information of the geologic history of urban areas is available.

GROUP	ORIGIN	CHARACTERISTICS
RESIDUAL	Mechanical Weathering	Rather thin deposits of coarse grained material
	Chemical Weathering	Mass property dependent on parent rock. Often clayey, depth to bedrock very irregular with possibilities of occurrence of large boulders.
TRANSPORTED	Fluvial (River-laid)	Deposits are very erratic but generally granular soils along river channel growing more fine around edges and cohesive in flood plains. Often find lenses of granular material in fines deposits.
	Lacustrine (Lake deposits)	Can be very regular in places and mostly fine. Where high seasonal fluctuation of inflowing water occurs get layer-

Table 2.2 GEOLOGIC ENVIRONMENTS AND MASS PROPERTIES*

*after Fenix and Scisson (3)

	ing or varving. Top strata may have high organic content.
Glacial	Very irregular deposits with grain size from very small to large boulder size. Only in areas influenced by glaciers.
Wind	Most important is loess which is a silt with cementation, very strong when no water is present but soft and compressible when wet.
Estuary (Coastal deposit)	Mixture of river and tidal deposits, very irregular vertical and horizontal boundaries. Appear near former coasts.

Table 2.2 Continued

2.5 Application of Geologic Information to Tunneling

Once geologic information concerning a proposed tunnel site has been accumulated, it must be assimilated by the designer so that concepts of what effect the geology will have on the tunnel design can be developed. A simple and helpful approach is to divide the geology into categories with similar behavioristic properties. This will be attempted in this section, but first a classification of ground behavior during tunneling described by Terzaghi (37) will be studied.

2.5.1 The Tunnelman's Ground Classification

A grouping of ground behavior with respect to an unsupported excavation in the ground, based solely on the standup time of the material, was presented by Terzaghi in 1950 for tunneling with timber supports. This classification and a verbal description with some of the associated ground parameters is presented in Table 2.3. With the replacement of standup time by the Standup Time Ratio,STR, of Equation 2.1, this description of ground behavior can be made applicable to any construction system.

Deere et al (10) linked USC classification and verbal soil descriptions to the behavioristic aspects of cohesionless soils. R_s in this table is defined as the unit standup time for a strip of tunnel roof of width L = 1 foot, and (see Figure 2.4):

$$R_s = t_s L \qquad (t_s \text{ in hours}) \qquad [2.6]$$

BEHAVIORAL DESCRIPTION	S.T.R.	VERBAL DESCRIPTION	ASSOCIATED GROUND PARAMETERS
Firm	>>1	heading can be advanced without roof support	strong cementation between particles, low plasticity, high OCR
Slowly Raveling	>1	flakes or chunks begin to fall out of roof and walls long after initial exposure	granular soils with some cohesion or cementation above water table
Fast Raveling	≃1	raveling process starts just minutes after exposure	same as slow raveling below ground water table
Cohesive Running	≃1	short period of fast raveling followed by general instability as soil pours into excavation	granular soils with no cementation but some apparent cohesion above water table
Running	<1	soil pours into excavation immediately upon opening until slope of material reaches angle of repose, $\phi_r \simeq 34°$	dry granular materials with no cementation or apparent cohesion
Flowing	<1	soil moves like a viscous liquid into excavation immediately when opened	$D_{10}>5 \times 10^{-3}$mm below water table, c<<water pressure
Squeezing	≤1	slow, plastic advance of material into excavation	soft or medium clay of high plasticity
Swelling	>1	slow advance of cohesive material into tunnel accompanied by large volume increase in material	P.I.>30, cohesive soils containing anhydrite

Table 2.3 TERZAGHI'S TUNNELMAN'S CLASSIFICATION

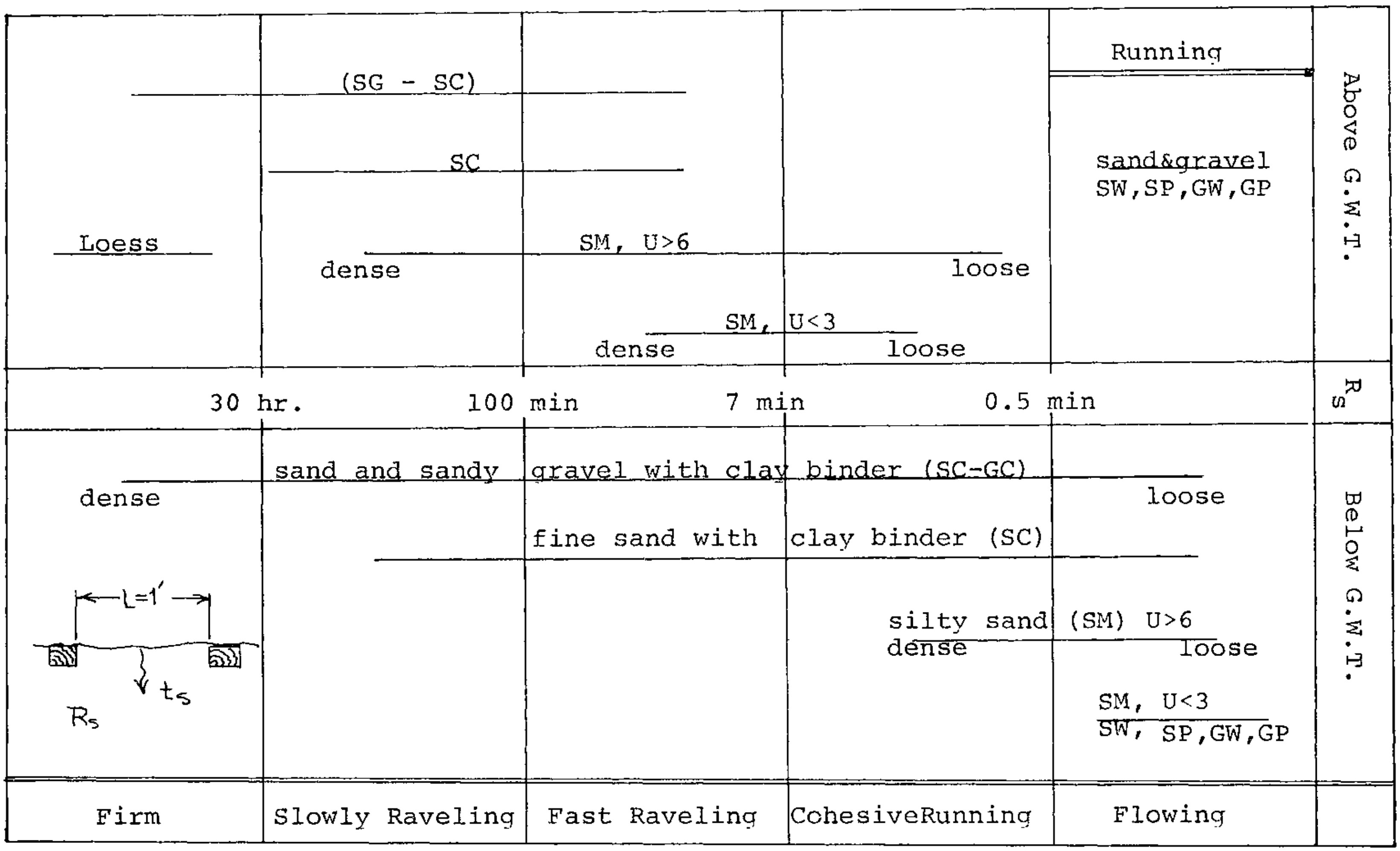


Table 2.4 CORRELATION OF TUNNELMAN'S CLASSIFICATION AND USC CLASSIFICATION after Deere et al (10)

Such a classification of course serves only the purpose of showing the distribution and relative behavior of many soils across the range of behavior. It does not take into account such factors as apparent cohesion above the G.W.T. and ignores cementation or true cohesion. It also shows a measure of standup time, R_s, which is independent of tunnel size and assumes no support. At least, however, it does exhibit a way of identifying potential behavior by an easily identifiable system which could be employed by an engineer at an early stage in the design. Such a system could be extended to plastic soils by dividing the category "Squeezing" (STR ≤ 1) into two, Squeezing (STR ≈ 1) and Very Soft Squeezing (STR<1) [see Fenix and Scisson (3.)] This would extend the Tunnelman's Ground Classification to cover the range of behavior of cohesive material in sufficient detail to give the designer an idea of potential behavior of clays and plastic silts.

With the addition of some of the parameters such as strength and permeability in Section 2.2, detail is added to the system through the ability to be more specific in defining tunnel behavior. Application of the Broms and Bennermark criteria, or the stability ratio of Equations 2.4, as a behavioristic description of the ground for soft ground tunneling has already been suggested by Wheby and Cikanek (42,) and some model testing has been performed by Attewell and Boden (4.) The adoption of such theoretical criteria as the stability ratio is a direct step in mating the identification of ground properties with some of the construction aspects of the tunnel design.

2.5.2 Identification of Ground Conditions

Thus far the discussion of ground conditions has focused on two separate factors that affect the tunneling process, the macroscopic geologic details, such as those listed in Table 2.1, and the engineering properties of separate, homogeneous materials. The relative importance of these two factors can be recognized by the engineer if the area geology can be determined to be either homogeneous or non-homogeneous relative to the tunnel construction. The appearance of a homogeneous ground can key the engineer into anticipating a tunnel construction system that is specialized to the ground parameters of the homogeneous geology. The identification of a non-homogeneous ground, however, can lead a designer to search for a tunnel construction system design which is more adaptable to variable ground conditions. Furthermore, in a homogeneous layer the optimum construction system design and total system cost can depend on whether the ground behaves primarily as a cohesive or cohesionless medium. The three identifiable types of ground are then:
1. Homogeneous Cohesive, 2. Homogeneous Cohesionless, 3. Non-Homogeneous.

The three types of geology that may be encountered, as described above, form a set of mutually exclusive and exhaustive states of what may be termed as a "geologic identification" parameter. These three types of ground may be subdivided further, (and in fact are so in Chapter 4) if the engineer decides that more states are necessary to adequately describe possible ground variations. This would occur for instance if he judged that one of the states had a dual nature with separate and significant cost consequences, (for example, dividing homogeneous ground into cohesive and cohesionless.) This is an example of how the states of parameters are formulated, the degree of specificity with which a parameter is described is dependent on the engineer's needs and resources.

The identification of the ground alone may be misleading for it is the relationship between ground and construction procedure which finally determines the behavior of the ground during construction. The next chapter therefore will review the construction process for both its effect on ground behavior and its sensitivity to geologic variation.

ELEMENTS OF TUNNEL CONSTRUCTION

3.1 INTRODUCTION

The construction of tunnels for mass transportation systems is not different from tunneling in general; but has limitations due both to the intended tunnel use and the geographic location. Such project parameters as depth, tunnel geometry, project length, and workshaft spacing are affected by the location of buildings and utilities, the vehicle dimensions, and passenger handling needs. These factors are important in the construction phase because their occurrence often is in the form of an obstacle to tunneling, either directly by physically impinging on the tunnel cross-section, or indirectly by being placed in jeopardy by possible ground/tunnel interaction. All these features combine to provide some unique aspects to the planning of construction strategies especially in relation to the occurrence of ground variation.

This chapter will first overview the construction of tunnels to identify cost elements and subsystems (components.) Next these subsystems will be evaluated according to their sensitivity to changing ground conditions. An attempt will be made to indicate how each subsystem can be divided into cost elements for use in the general tunnel cost model which will be developed in Chapter 4. Finally, those aspects of tunnel construction which are ground related and impact the tunnel's environment will be examined for pertinence with respect to the ground model. Several recent reports by Bechtel, Inc. (35,) Fenix and Scisson, Inc. (3,) and Mayo (22) have dealt extensively with tunnel construction subsystems and the resultant implications for cost and time. This chapter will only outline the major components as necessary to describe application of the probabilistic approach.

3.2 Cost Components of Soft Ground Tunneling

The construction of mass transportation tunnel is usually preceded by the completion of a workshaft to the elevation of the line tunnel. This workshaft is needed to allow access of machinery, men, and materials to the working face, and its location is usually determined by the positioning of stations and ventilation shafts. The cost for the excavation of the working shaft and the cost for assembling the tunneling equipment at the site, can be substantial; and must be included in the cost evaluation of the project. These costs do somewhat vary with ground conditions. The contractor may want to change his strategy with ground variations which could call for additional workshafts to open several working faces or the necessity of supplying additional equipment to the site. The occurrence of these considerations can and should be accounted for through application of the probabilistic approach, and that is why they are mentioned here. Since the intent is just to present the method and show its applicability, however, these factors will be assumed to be ground invariable in order to simplify the presentation.

The tunnel line construction accounts for approximately 70% of the total cost of a tunneling project and the purchase of the tunnel liner (not installed) another 15-20% (35.) Given a particular set of ground conditions upon which the design is based, theoretically all of the elements that comprise the above 85-90% of the total tunnel cost are subject to change, depending on the cost component in question.

Cost components here refer to the subsystems which comprise the actual tunneling process, and which were divided, in the Fenix and Scisson report (3,) into three categories: 1) excavation, 2) support (temporary and permanent,) and 3) materials handling. The definition of support is quite broad, including stabilization of the ground for excavation, temporary ground support before placement of permanent liner, and application of the permanent liner itself. For the purpose of dividing the tunneling process into subsystems (or cost components) support here will be divided into two groups. One group, ground stabilization, will refer to any method of changing the ground properties so that tunneling can proceed; while the other, ground support, will mean any method of physically bearing the stresses imposed by the ground. The former group will, therefore, in general apply to any method which changes the strength of the material, S_u in the denominator of Equations 2.4. The latter subsystem acts to support the ground load, γz, of the numerator in those equations. (The exception is compressed air, but the difference will become evident in the explanation later.) The revised subsystems then, which will be considered to comprise the tunneling process, are: 1) excavation, 2) ground stabilization, 3) ground support, and 4) materials handling. Each of these components is comprised of cost elements which will be assumed to fall into three general categories; material costs, labor costs, and operating cost.

The material cost refers to the initial cost of equipment, structural elements, and supplies necessary to construct the tunnel. These costs depend on the geometry of the tunnel, especially the length and the construction procedure which is to be employed. The rate at which the tunnel progresses has little to no effect on these costs unless there exist supply contract clauses which set a time limit on the applicability of a given material cost. In this case substantial construction delays could lead to cost escalation. An example would be if an agreement between contractor and supplier fixed the cost of cement up to a certain date. If construction delays forced purchase after that date, prices for the cement could go up.

Usually material costs do not change very much with ground condition variation, unless the geologic change is significant enough to force the contractor to employ methods of construction for which he did not plan. If a ground stabilization technique fails and another must be employed, additional supplies and equipment could be required which would result in additional costs. Such changes would also probably result in cost penalties because premium prices could be expected for additional work negotiated after construction commencement.

Labor costs are not fixed by the length of the tunnel, rather they are dependent upon the time required to complete the project, which is a function both of length and rate-of-advance (ROA) of the tunnel. The labor cost could vary two ways. The size of a tunnel construction crew could change depending upon the construction method employed and the difficulty of applying a particular construction strategy to a particular set of ground conditions; or a change in the system ROA could lead to a proportional change in total labor cost over the length of the project. The cost of the construction crew is in itself a variable dependent on the construction method employed. The average crew cost increases with a corresponding increase in both degree of specialization required by the construction procedure and the hazard related to a particular tunnel method. (These are the only two crew cost variations which are really dependent on ground conditions, the factors related to labor supply and geography are assumed part of To as explained in section 1.2.)

The sensitivity of labor cost to ROA and the dependence of ROA on ground type makes this particular cost category particularly affected by changes in ground conditions. The change in labor cost for a variation in ground condition can be estimated from a prediction of the changes in average ROA, crew size, and average crew cost. If the tunnel construction is delayed for any length of time and the crew is laid off (so that labor costs are not incurred over that time period,) a cost penalty is still paid because of the reaction time from construction delay to crew layoff (crews may be kept through several days of no work) and the problems of rehiring the crew and restarting the operation.(The so-called learning curve for example which refers to the increase in ROA from the commencement or restart of the construction operation as the work crew "learns" its assignments and begins to coordinate its functions. The period of this learning curve could vary from several days to several weeks.)

Operating costs include the cost of fuel and utilities needed by the tunneling machinery and maintenance costs to keep the machinery operating. The analysis of these costs is very complex because of their dependency on the project length, system ROA, and each component rate of operation. The cost of operating a machine excavator excluding labor cost is proportional to the rate at which the excavator is advanced, which is some fraction of the ROA of the system. (The relation of excavator advance to system advance is explained in the next section.) The faster the component rate of operation, the higher the operating cost; but the less time the component is needed to operate over the length of the project. This is illustrated in the plot of operating cost ($) versus ROA in Figure 3.1. As the ROA increases, the cost of operating the tunneling "machinery" increases (unit operating cost) while the cost of operating over the fixed length of tunnel decreases because the time of operation decreases. These costs in the model analysis will be assumed to counterbalance each other so that the sum of the two curves in Figure 3.1, or total operating cost, is constant. In most design cases no data will be available to evaluate these factors anyway; but in those cases where data on operating costs in relation to ground conditions is available they, of course, should be included.

Another facet of operating costs is the cost associated with operating a system which is independent of ROA but whose needs may vary with ground conditions. Ground stabili-

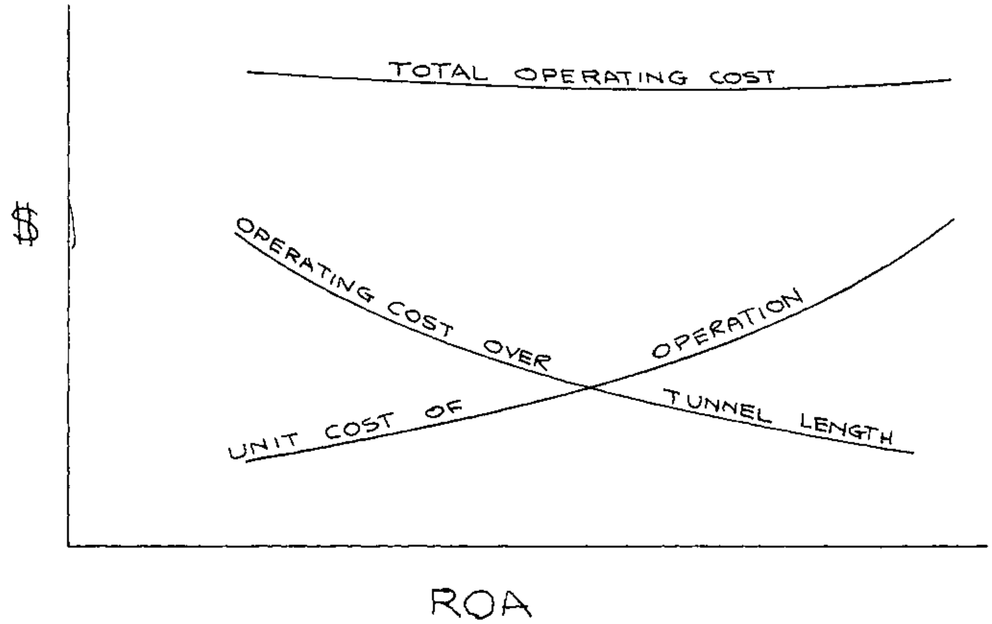


Figure 3.1 OPERATING COSTS AS A FUNCTION OF RATE OF ADVANCE

zation methods which operate from the surface are examples of such systems whose operating costs are nearly independent of rate of advance, but can change substantially with ground conditions.

3.3 Description of Tunneling Subsystems

3.3.1 Excavation

There are as many different excavation systems as there are tools with which to dig. These can range from hand tools such as spades and picks to sophisticated digging machines called moles; and the characteristics of these methods depend on the excavator and the ground conditions. For purposes of analysis these excavation systems can be divided into two groups, hand mining and machine mining. Both methods can be accomplished from within a shield, by forepoling, or, where the ground is firm and can be left unsupported for a period of time (or if employing such tunneling approaches as the New Austrian Tunneling Technique which does not need a shield,) without support.

Hand mining, which will be defined here as any method not employing a tunnel boring machine or a shield mounted ripper bucket, can use power hand tools (power spades and power picks) or even independent, vehicle mounted excavation equipment. The key to the difference between hand mining and tunneling machines is that the tools used in handmining are completely independent of any wall support; tunneling machines as defined here are attached to the shields or are within enclosures which resemble shields. Due to the independence of the tools of hand mining from the tunnel, they can be employed again in other projects, and in general cost less initially than tunnel machines. Cost of the excavation component when hand mining is employed is very much dependent on the cost of labor which, as mentioned in the previous section is a function of the ROA.

Versatility is the major advantage of hand mining, it can be used with almost all forms of stabilization and support as long as the excavation can be kept dewatered, and stable enough to prevent large ground movements. The occurrence of face instabilities in adverse grounds can be counteracted through use of face breasting and excavation of only small areas of the face at any one time. Where very stiff cohesive or well-cemented materials are met, progress can be slowed due to difficulty with actual excavation itself. Boulders also slow the ROA because of the increased time necessary to break them up and haul them away; but such unexpected occurrences can be met. Hand mining is the most labor intensive of all the excavation techniques and is therefore most sensitive in general to the workmanship and experience of the tunneling crew. Conditions which adversely affect the workmen, such as water inflows through the tunnel will slow progress. Hazardous conditions such as work in compressed air, which increase the unit cost of labor have a large effect on the cost of hand mining because of the dependence on labor. The cycle of construction itself is very flexible and amenable to change and adaptation.

Mechanized excavation methods can be incorporated with shields to become so-called moles or tunneling machines. In stable, or stabilized, soils these machines can be employed using an open wheel, cutter arms, or a ripper bucket depending on the properties of the material. For unstable soils, a means of face breasting, or some other method of closing off or supporting the face, may be employed as stabilization. Shields with the entire face completely closed off except for a door through which the muck can be collected or extruded have been employed in very soft soils, as in San Francisco according to Kuesel (20,) to prevent soil intrusion at the face.

The use of a shield has several functions. The shield serves as immediate protection for the walls as excavation is continued. Most importantly, perhaps, the shield provides protection for the workmen. The initial step in the construction cycle is the shoving forward of the shield into the soil, with reactions for the thrusting movement usually on the completed lining. Sometimes, in very hard ground or when obstructions are met, some pre-excavation must be performed ahead of the shield. Where the ground ahead is soft or sensitive to compression, excavation must be carried on with the shove.

Tunneling machines can reduce the labor requirements for excavating a unit volume of tunnel, and they do increase ROA under the proper ground conditions; but these advantages come at a cost. The first consideration is the increased capital investment in equipment, which is usually depreciated over the length of the job. The ROA must be increased substantially over the whole length of the project to offset the higher initial cost. Next, the excavation process becomes much more sensitive to variations in ground conditions when a machine is used. A local instability causing a run at the face could partially bury the machine. Due to the need for freeing and mending the machine, the delay in time is normally much greater than if the delay occurred during hand mining. Part of this time delay is also due to the lack of space availability at the face. The excavating equipment normally fills a large portion of the space and movements therefore are constricted. Dealing with an obstruction such as a boulder or encounter of the rock surface tends to become a substantial problem. With a completely closed face the complexity of dealing with an obstruction becomes impossible, and a new excavation strategy has to be devised for that segment; usually at very great cost.

The problems that tunnel machines face are symptomatic of the premium of working space and the difficulty of altering operations at the face. Any work that must be carried on ahead of the face, such as the excavation of rock around the periphery of the excavation or the removal of boulders that only partially intrude into the cross-section, is very difficult. The interaction with the other subsystems subsequently becomes even more difficult. Due to the lack of face availability, grouting at the face must compete with the excavator, the mucker, and the liner erector for space. The longer these other cycles take, the less time the machine is being utilized, and the more the degeneration of ROA from the design theoretical of the excavator. Interspersed with all these requirements, is the need for periodic maintenance of machinery. A maintenance cycle should be added to the construction process or long delays will occur from non-scheduled maintenance operations.

Under ideal (predictable) conditions, mechanized excavation systems can far outperform hand operations. The system ROA of a well-planned machine mined tunnel approaches the ROA of the excavator, which means that the other components comprise less of the construction cycle independent of the excavation. An interesting test of comparative performance was recently demonstrated in Toronto, recorded by Mayo (22,) where twin, parallel tunnels of 17.5 foot diameter were constructed using different excavation techniques. One tunnel was constructed using hand methods within a shield, while the other was done using a mechanical excavator within a shield. The rate of advance for the hand excavated tunnel averaged 9 linear feet per 24 hour (three shift) day; while the machine averaged 20 linear feet per 18 hour day. Of course, with the occurrence of adverse conditions, ROA may have been more equal for each tunnel, with hand operations being more cost effective.

3.3.2 The Ground Stabilization Subsystem

The purpose of the ground stabilization subsystem is to alter the properties of the ground so that the characteristic behavior is improved enough to allow successful excavation and support. The attempt is to convert a system where standup time ratio (STR)$\leq$1 to one where STR is at all times >1 through the modification of material properties. The three major forms of stabilization are: dewatering, compressed air, and grouting. There are other, more exotic, systems such as freezing and electro-osmotic stabilization, but for reasons of cost and environmental effect these techniques are not widely used in cities in the U.S.A. Their incorporation is certainly not excluded from this approach, the designer may incorporate any system whose performance he feels can be predicted with reasonable certainty. These methods will simply not be discussed here because of their limited history in the U.S.

The desirable ground stabilization system for a particular purpose is dependent on the cost of the system to perform its function effectively, and the characteristics of the ground to be stabilized. Table 3.1 indicates the range of each type of stabilization system as a function of the mass permeability of a soil layer. Of course the capacity of any material to be stabilized is a function of other soil properties, including the distribution of the permeability through the layer and grain size distribution; and the relative confidence in the system to perform successfully should decrease rapidly as the limits of the functional range of the system are reached. Often one technique will be mobilized and readied as a back up for the primary system where there is uncertainty concerning the successful performance of the first system.

Dewatering is performed usually from the ground surface in advance of the tunnel construction and, as is evident from Table 3.1, it is effective primarily in granular soils. The lowering of the ground water table increases the strength of the soil by increasing the frictional component of strength and improves stability by relieving the intergranular pore water pressure. The effect on the behavior of the material is to increase the standup time through an overall increase in stability, see Figure 3.2.

The cost of dewatering is dependent on the quantity of water to be pumped and the permeability of the ground, both factors being related. The cost factors involved include the equipment needed, (pumps and wells) installation, and the operation costs. A thorough discussion of predrainage was presented by Powers (28) including variations in costs dependent on ground parameters. It is often difficult to predict the success of a dewatering operation because of the variability of geologic factors. The consequences of an inadequacy in the system can range from relatively minor to very serious. At the least there may be some increase in the water in the excavation, causing some stability problems,

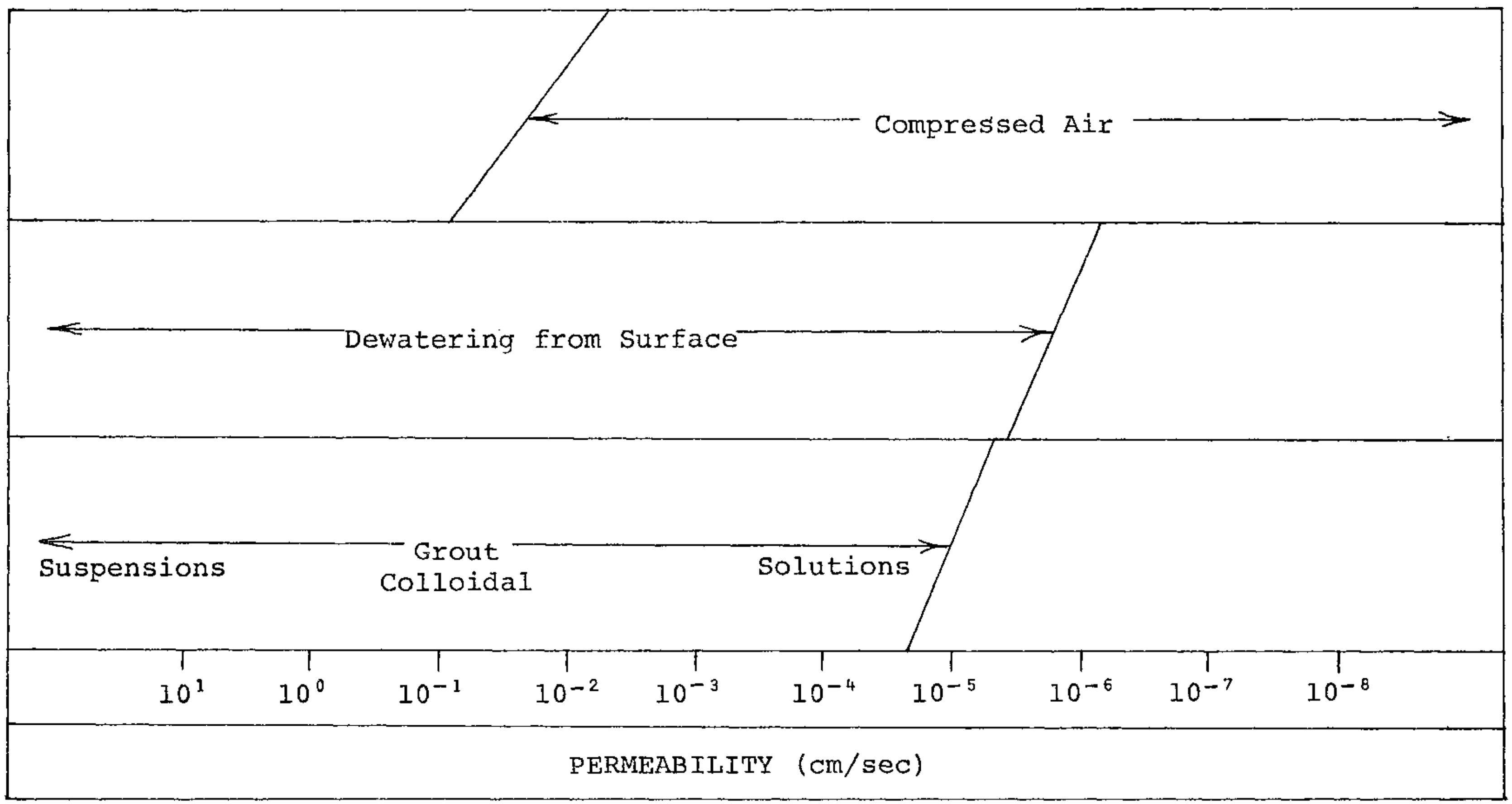


Table 3.1 GROUND STABILIZATION METHODS AS A FUNCTION OF PERMEABILITY

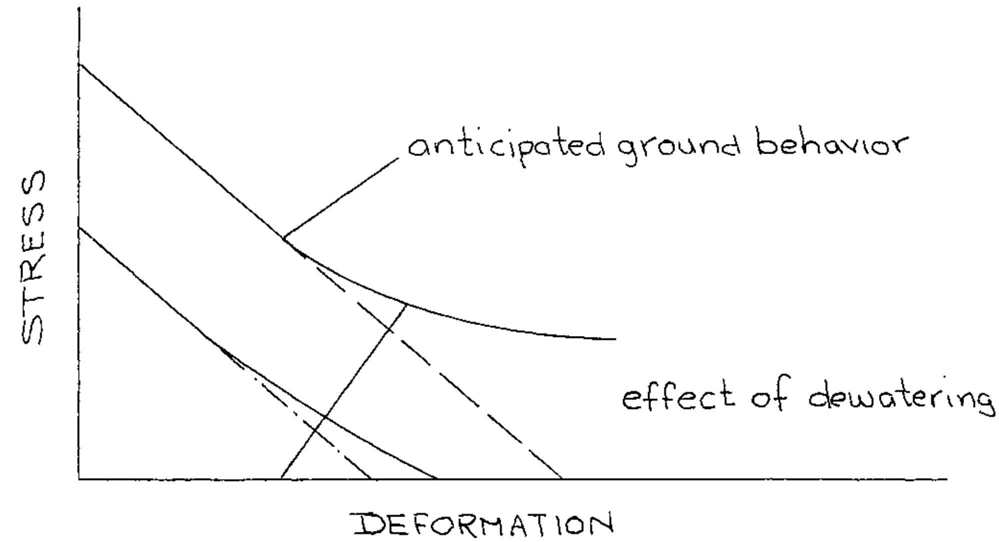


Figure 3.2 THE EFFECTS OF DEWATERING ON THE CHARACTERISTIC BEHAVIOR OF A SOIL

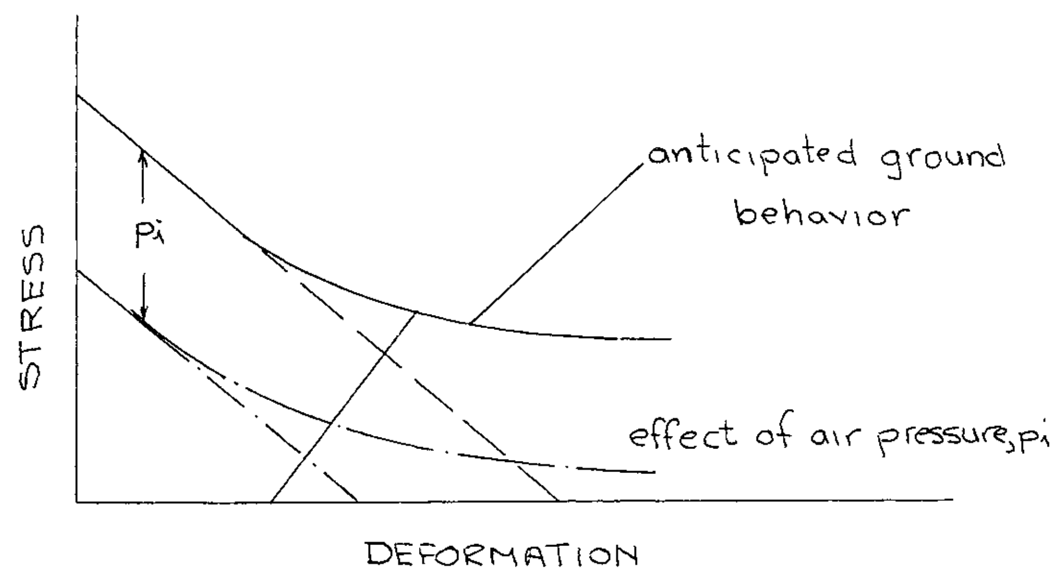


Figure 3.3 THE EFFECTS OF COMPRESSED AIR ON THE CHARACTERISTIC BEHAVIOR OF A SOIL

but putting a load on the tunnel's internal dewatering system. On the other hand pockets of undewatered material can cause face collapse and excessive ground movements behind the shield.

The suppression of the water table around the excavation can have serious effects on the surroundings, especially in the urban environment. The most serious problem is the possibility of ground settlements where the water table is lowered through soft or compressible soils. In these materials, the decrease in porewater pressure leads to an increase in $\bar{\sigma}$ sufficient to consolidate the material. Other problems could be encountered in areas where the water table is lowered, exposing timber piles which may begin to degrade. The depressed water level could also lead to the intrusion of water from a nearby polluted or otherwise unfit source. In these cases, the effects of dewatering can be confined to a local area around the tunnel by recharge; but this adds yet another variable to the success of the subsystem and does not help structures directly over the tunnel still in the dewatered zone.

Compressed air can be used to increase the internal pressure in the tunnel, thereby decreasing the stability ratio (Equations 2.4) and improving the characteristic behavior of the material as shown in Figure 3.3. In granular soils, the pressure of the compressed air can counteract the water pressure at the tunnel face and force STR >1 so that excavation can be accomplished more easily. Sometimes the addition of just a small amount of air pressure can lead to the difference between a material difficult to work with, STR ≈ 1, and one where STR >1 and the material is easy to work with. This difference can result in an appreciable increase in ROA.

The employment of compressed air on a project means the acquisition and deployment of a large compressed air plant and the inclusion of air locks for both materials and manpower in the tunnel. These features are expensive and increase the complexity of the construction operations leading to an increase in things to go wrong. A back up compressor is required in case the first compressed air plant fails, for the sudden disappearance of p_i in the tunnel could be catastrophic. The workmen also face the risks of sudden decompression which can mean affliction with the bends or even bone necrosis.

Compressed air is often used in conjunction with dewatering so that lower air pressures are required. Air pressures under a head of approximately 100 feet of water would be too great for workmen, so there is a depth limitation. At the same time, the stability ratio is not constant over the entire face. Sometimes, in order to increase the stability at the invert the pressure at the crown is enough to force a reverse flow of water in granular soils, drying out the material and causing local instability to occur. Decompression can occur when permeable layers which allow the air to escape are encountered. A more complete discussion of the pitfalls in compressed air can be found in Mayo (22.)

The primary disadvantage of compressed air is the cost of labor which can increase as much as several times when compression is employed as a stabilization. Jacoby (19) points out that an increase in labor costs of 250% was avoided by choosing predrainage instead of compressed air for the construction of a New York sewer tunnel. In this case, even though the ground was variable, dewatering was still chosen as the stabilization system although a compressed air plant was set up as a back up in case predrainage failed to stabilize the uncemented sands and silts. In many cases it may be more economical to set up a compressed plant for those short sections where it is the only stabilization which will succeed; and attempt to excavate the remainder of the project with an alternate system in free air.

The effect of grouting is to increase the frictional component of strength and provide at least temporarily a cementation or apparent cohesion which may stabilize the material as shown in Figure 3.4. This stabilization is accomplished through the injection of cement or various chemicals into the soil ahead of an excavation. Grouting can be accomplished either from the surface prior to excavation or it can be interjected into the construction cycle and performed from the face. In this way it is more flexible than dewatering in that it can be employed, with an associated time penalty, of course, where conditions at the surface do not permit the drilling of holes.

The choice of the type of grout to use is dependent to a great extent on soil conditions, especially permeability. The selection of grouts, including some discussion of relative costs, was thoroughly assessed by Anderson and McCusker (1) and Haffen and Janin (16.) The variation of soil conditions and especially the variability of ground water flow through the soil can lead to a great deal of uncertainty as to the effectiveness of the grout as a stabilizer.The non-uniform set of the grout in zones of greater permeability can lead to instability upon excavation in those less permeable zones where the grout did not take. The flow of ground water can also cause dispersion of the grout before it has time to set and lead to a strength much less than anticipated in the material. Even with a fast setting grout, its stabilizing effect will deteriorate with time as a function of the action of the ground water.

The cost of grout is proportional to the area to be grouted (a function of length) and the soil type. Where grouting is accomplished from within the tunnel it also becomes ROA dependent. The major costs are the supplies and installation of boreholes, with cost in general increasing as the permeability decreases.

The selection of a ground stabilization system is critical to the success of the tunnel construction and all potential uncertainties must be evaluated carefully. If there is a possibility of failure which could lead to catastrophic occurrences, then viable alternatives, perhaps in the form of a combination of methods or a change in excavation or support strategy, must be implementable. If not then the consequence is an abandonment of the excavation.

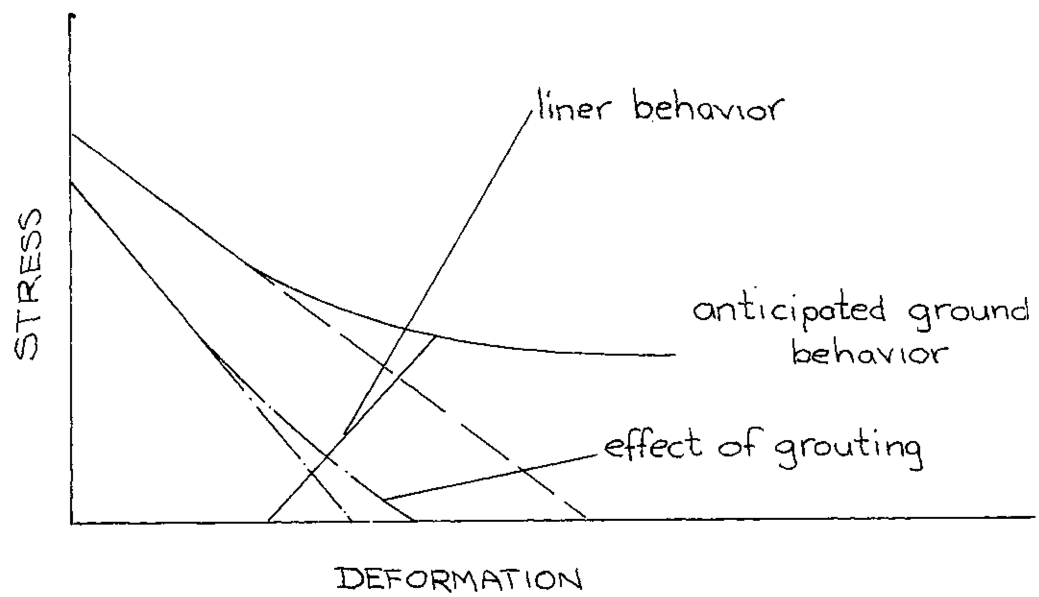


Figure 3.4 THE EFFECTS OF GROUTING ON THE CHARACTERISTIC BEHAVIOR OF A SOIL

3.3.3 Tunnel Support Subsystem

A study of any of the characteristic curves presented previously will indicate that the opening deforms as stress is relieved around the excavated cavity. One general characteristic of soft ground is that it will continue to deform until some support is provided (this is not quite true for firm ground which tends to behave as soft rock; Deere et al (10) present a good discussion of the differences between soft and hard ground.) The support resists stresses in the soil at the time of application which are usually somewhat less than the in situ stresses; but with time can increase (see Figure 2.5) The long term adequacy of support is solely dependent on ground conditions and if there is uncertainty, must be evaluated. Short term loads on supports usually pose no problem, but the time from excavation to liner erection can be critical.

The shield is the first support which is provided the tunnel walls; but as it is moved forward, the soil over the tail is left exposed and unsupported. If the soil is of sufficient stability to display only elastic behavior during the period while the soil is unsupported, the liner can be expanded directly against the ground, preventing further movement. The time required for this operation, the support application time is:

$$t_a = (L_s/r_j) + to \qquad [3.1]$$

Where L_s = the length of one liner segment, r_j = rate shield advances into soil, and to = time to assemble and erect one liner segment. Since the liner may be partially or fully assembled before the shield is advanced, to is really more accurately described as measuring the time between withdrawal of temporary support and application of final support. Only in firm ground which is unsupported during excavation will to in reality equal the time of assembly and erection. It can be seen that variation in this time will be fairly insensitive to ground conditions, except when $r_j \to 0$ due to unforeseen delays or obstructions. The skill and productivity of the crew is however a variable that must be taken into account especially as STR $\to 1$.

In soils where STR ≤ 1 when unsupported, t_a must be decreased. The most effective means of doing this is by decreasing to, although an increase in r_j will also help. To decrease to, the liner can be erected in the tail of the shield and expanded as soon as the segment is clear of the trailing edge (this is usually the case.) In ground/construction systems where STR <1 (t_s is very small) the liner can be completely erected within the shield and the annular void, created by the difference in outside diameters of the shield and liner, filled with a grout or pea gravel. In this case, to becomes very small, though finite, and increases or decreases in L_s and r_j become very important.

Another function of the lining is to act as a water proofing during the life of the tunnel. If the liner is not of itself water proof, then some means of sealing it must be provided and added into the work cycle. Deformation after this water stop is in place can lead to rupture of the seal and the leakage of water into the tunnel. Peck et al (27) described in detail design procedures and considerations for tunnel liners.

3.3.4 Materials Handling Subsystem

The flow of materials into and out of the tunnel during construction is a problem in coordination and space availability. Liner materials must be brought into the tunnel, and utilities, electricity and air, must also be provided. Soil, water, and manpower must be transported between the surface and face. In many respects materials handling is not a subsystem in itself, but is an extension of the other components of tunnel construction. Even though a cost can be estimated, such as for the installation and operation of a conveyor or a rail mucking system, the actual operation of this subsystem is a function mainly of the other components of tunnel construction rather than the interaction with the ground.

Some connection with ground conditions does exist. The state of the muck as excavated can be influential on the type of mucking system chosen. Very wet muck, for instance, would be very difficult to remove by conveyor, just as large pieces of boulders also may give a conveyor trouble. The ROA is also somewhat related to the ability of the system to handle the quantity of material excavated and keep pace with the advance of the face itself (i.e. the muck train must also be advanced as excavation proceeds.) Usually, however, the influence of the materials handling subsystem is felt only in a negative aspect, as when it fails. Jacoby (19) reports that the periodic failure of the materials handling system in the New York sewer tunnel construction was the major factor in excavator downtime. A comprehensive examination of materials handling systems was accomplished by Fenix and Scisson, Inc. (3) and Fillip (14.)

3.4 Effects of Construction on the Environment

One of the positive features of tunnel construction versus an open cut and cover excavation, is that there is very little adverse effect on the environment. Interaction with the surface is confined to the work shafts, and these areas are small in comparison to an open length of tunnel so the amounts of traffic interference, noise, and air pollution are significantly reduced. In fact the two most important impacts are the effects tunnel construction has on the water table and the settlement of the surface. Negative effects on the water table have been discussed briefly in a previous section, the rest of this section will be concerned with surface settlement.

A considerable amount has been written recently about the prediction of surface subsidence including work by Peck (25) and Schmidt (29;) and several recent case histories have been reviewed by Schmidt (31,) Hansmire (17,) and Stroh (34.) It is beyond the scope of this paper to delve into the detailed analysis of surface settlements; but the problem is a weighty and costly one for the designer, therefore some general aspects of the interrelationship of settlements and the ground/tunnel construction system will be discussed here.

The questions of primary importance to the designer concern the effect of surface subsidence due to tunnel construction on structures surrounding the tunnel; and the possibility of remedial or preventative measures due to this subsidence which may add significantly to the cost of the tunnel. The most costly damage is frequently to buildings and is the result of differential settlements due to the subsidence caused by tunnel construction. Frequently less noticed, but often costly and possibly catastrophic, is disturbance to buried utilities. Settlements under utilities are frequently masked by the paved surface of streets (20) which bridges the subsidence troughs. Utilities which might have been in place for many years, and whose resistance to bending has been weakened, may break. The most serious util-

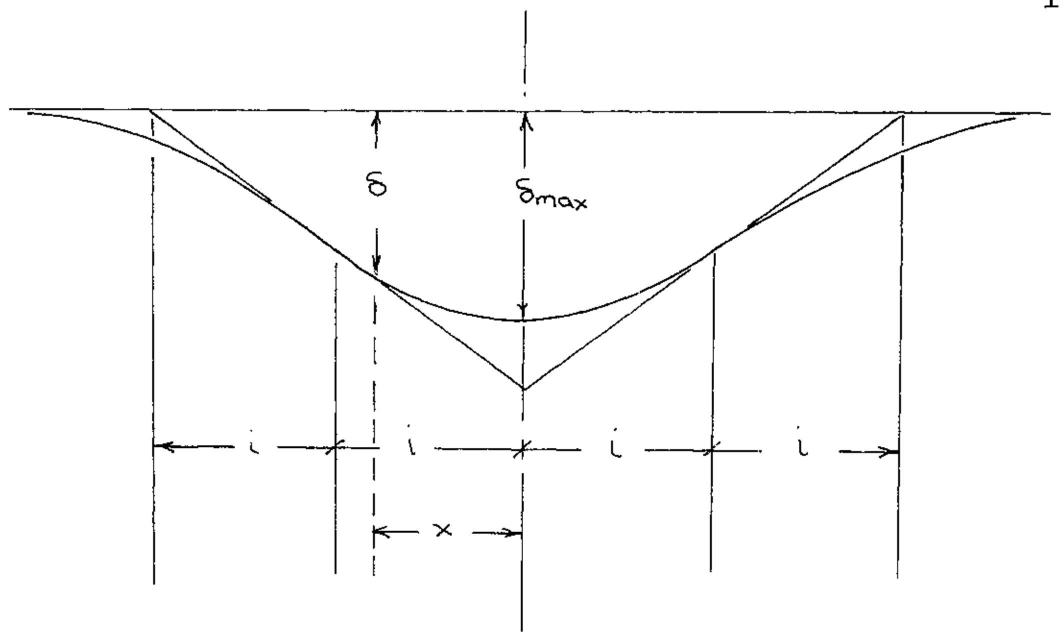


Figure 3.5 GAUSSIAN DISTRIBUTION OF SETTLEMENT

ity breaks, from the point of view of effect on the tunnel, are breaks in gas lines and water lines.

One of the major causes of surface settlements caused by tunnels is ground volume lost at the tunnel level itself during excavation. This ground loss can occur at the face of the excavation where the amount lost will increase with increased instability of the face. The source of this lost ground can be better understood if one examines Figure 2.3 as the vertical displacement of a hypothetical point at the crown elevation of the tunnel. This point begins to move downward with respect to the tunnel, even before the leading edge of the shield reaches it. When the shield does arrive, this point has displaced well into the cross-section of tunnel to be excavated. As the shield moves through the perpendicular plane that this point was on, the surrounding ground is further disturbed by friction of the shield. As a result the soil will encroach into the annular void at a greater rate. The final region of ground loss is the elastic deformation of the tunnel liner. More ground loss can be anticipated on curved sections of tunnel and in materials where guidance of the shield is a problem due to plowing of the shield causing an annulus larger than just the difference in the outside radii of the shield and liner.

The settlement on a free ground surface has been suggested by Schmidt (29) as resembling the form of the distribution function, see Figure 3.5. The approximate extent of the settlement, 4i, is then calculated from:

$$i/a = 1.0(z_o/2a)^{0.8} \qquad [3.2]$$

Where a $\equiv$ tunnel radius and z_o $\equiv$ the depth to tunnel centerline, and the distribution function is:

$$\delta = \delta_{max} \exp\{-x^2/2i^2\} \qquad [3.3]$$

δ is the settlement at any point x away from the centerline of the trough where δ_{max}, the maximum settlement occurs. The volume of the trough, V_s, which is approximately equal to the volume of ground lost at the excavation, provided no volume change has occurred in the layers of material above the crown, is approximately predicted by:

$$V_s \simeq 2.5 \cdot i(\delta_{max}) \qquad [3.4]$$

According to Peck (25,) assuming good workmanship, the volume of the settlement trough should amount to approximately 1% of the excavated volume of the tunnel.

The assumption of good workmanship in the above analysis also assumes that STR>>1 and that very little material is lost other than in the tail voids. Of course no plastic deformation is assumed. As STR→1 one can assume that V_s will increase. Also it should be noted that theoretically $i \propto z$, while $\delta_{max} \propto z$; which means as the depth of the tunnel increases the extent of the surface subsidence increases but the maximum value of subsidence decreases.

Another assumption which must be carefully evaluated is that of no volume changes in the ground, which is not true for all soils. Certainly in dense sands one would expect some volume increase, while in loose sands disturbance may cause an increase in relative density, and in these soils one would expect deviations from the above theory. Some deviation should also be expected in soft clays, and the presence of soft compressible layers are always suspect. In any case the data for correlating subsidence in real situations is accumulating and it is possible to make better predictions as more experience is garnered.

The translation of surface subsidence to buildings for the purpose of determining additional support requirements is more difficult, as there is no available theory to predict building damage. Some empirical correlation between building damage and settlement troughs in free ground has been presented by Chambosse (9.) Major damage has been recorded in buildings where foundations have intersected the plastic zone around the tunnel cavity. Common sense would dictate that where foundations are founded in material above the tunnel, the imposition of building loads and changes in stresses due to the movement of material below may lead to consolidation in layers even where subsidence due to the tunnel construction would have been negligible. In this area more field data is needed before truly definitive guidelines can be established.

FORMULATION OF PROBABILISTIC APPROACH

4.1 SYNTHESIS OF APPROACH

Probabilistic and decision-making tools have been introduced and the general approach to tunnel design outlined. This approach relied on three basic steps: the formulation of a cost model of the tunneling process, modeling the ground probabilistically using the decision tree framework, and design evaluation using equivalent monetary values as decision making criteria. The relation between ground conditions and the tunneling process has been reviewed with respect to the consequences in cost variation of ground changes. It is now time to synthesize this knowledge of ground conditions, the tunneling process, and decision theory, and apply them to the design of soft ground tunnels.

The next section of this chapter will further define the general tunnel cost model which was introduced in Section 1.3.1. The object will be to show how costs can be developed for various tunnel design schemes as cost consequences of the variation of ground conditions. The specific configuration of the geologic decision tree model, with parameters and states selected to reflect soft ground tunneling in the urban environment, will then be presented. To conclude, some reflections on the application of the approach to tunneling design will be presented.

4.2 TUNNEL COST MODEL

The general tunnel cost model presented in Section 1.3.1 defined the cost of a tunnel project by two factors, To which for a given length of tunnel does not vary as the ground conditions change, and B_j the cost of the j^{th} element which is sensitive to changes in ground conditions. Mathematically this model is just the sum of these two elements:

$$T = To + \sum_{j=1}^{m} B_j \qquad [1.2d]$$

The tunneling process was separated into four components in Chapter 3; excavation, ground stabilization, tunnel support, and materials handling. Considering these to be the four basic cost elements, j must equal four. These components can be broken down further into separate cost elements which can be categorized as either material, labor or operating costs. Therefore, the ground variable cost of the j^{th} element, B_j, can be further subdivided into a cost of additional materials due to ground variations, ξ_1; the cost change in labor due to changes in ground conditions, ξ_2; and the fluctuation in operating costs due to these changes in ground condition, ξ_3. The total cost of the ground variable elements of B_j is then the sum of these three costs:

$$B_j = \xi_1 + \xi_2 + \xi_3$$

or

$$B_j = \sum_{n=1}^{3} \xi_n \qquad [4.1]$$

For the whole tunnel system, the ground variable cost elements described by Equation 4.1 can be summed over the four subsystems to arrive at the total cost B of all the ground variable elements.

$$B = \sum_{j=1}^{4} B_j = \sum_{j=1}^{4} (\xi_1 + \xi_2 + \xi_3)_j$$

or

$$B = \sum_{j=1}^{4} \sum_{n=1}^{3} \xi_{nj} \qquad [4.2]$$

The first cost element, ξ_1, is representative of the material costs which are sensitive to geologic variation. Material costs are dependent on project length and, since most materials are purchased in advance, are affected by ground variations only in extreme situations. These unusual occasions arise when ground changes necessitate modifications to construction strategies requiring either the purchase of additional equipment or the modification of existing equipment. ξ_1 then will be equal to the sum, X, of the costs for either additional equipment or equipment modifications necessitated by ground variations, or:

$$\xi_1 = X \qquad [4.3]$$

Labor costs are more complex to evaluate than material costs, because labor costs are a function of time to complete a project, crew size, and average crew wages; all of which are sensitive to variation in geology. Labor costs for the tunnel line construction can be all considered to be ground variable since they are all dependent on the temporal length of the project. If the cost of labor were Y (\$/day;) ξ_2, the labor ground variable cost, would be equal to Y times the number of days, t, it takes to complete the project, or:

$$\xi_2 = Yt \qquad [4.4]$$

in dollars. The temporal length of the project in days, t, is equal to the length of the project, L, divided by the average daily ROA so that:

$$\xi_2 = Y(L/ROA) \qquad [4.5]$$

Operating costs vary in the same manner as labor costs so that the operating cost component of the j^{th} subsystem, ξ_3, is equal to an operating cost time factor Z times the length over the ROA:

$$\xi_3 = Z(L/ROA) \qquad [4.6]$$

The cost of the ground variable elements of the j^{th} subsystem, B_j, then is equal to Equation 4.1, which when combined with Equations 4.3, 4.5, and 4.6 is:

$$B_j = \sum_{n=1}^{3} \xi_n = \xi_1 + \xi_2 + \xi_3 =$$

$$X + (L/ROA)(Y + Z) \qquad [4.7]$$

The total of ground variable cost elements summed over the project, B, is expressed by Equation 4.2 and is equal to:

$$B = \sum_{j=1}^{4} \sum_{n=1}^{3} \xi_{nj} =$$

$$\sum_{j=1}^{4} [X_j + (L/ROA)(Y_j + Z_j)] \qquad [4.8]$$

Where X_j, Y_j, and Z_j are the total material cost changes, labor rate, and operating cost rate respectively for the j^{th} component of the tunnel system.

To obtain B; X, Y, and Z must be summed for each ground change over the length of the project. If the alignment were divided into segments which could be modeled by one decision tree, i.e. all ground conditions could be defined by several parameters consisting of mutually exclusive and exhaustive states; then the model could describe the total cost, T_ℓ, of a segment of length ℓ, by combining Equations 1.2d and 4.8:

$$T_\ell = T_{o\ell} + \sum_{j=1}^{4} X_j + (\ell/ROA)(Y_j + Z_j) \qquad [4.9]$$

Where $T_{o\ell}$ is the cost of non-ground variable elements for the particular segment of tunnel and the remainder of the equation is just equal to B, the cost of ground variable elements from Equation 4.8 for the segment.

The problem with applying Equation 4.9 is that it may be difficult to separate a given cost element, say support erection labor cost for example, into a part which is ground variable and one which is ground invariable. A remedy for this difficulty is to slightly redefine both $T_{o\ell}$ and B. If a given set of ground parameters are assumed for a segment of length ℓ and a cost T_ℓ is computed, then this cost can be assumed the nonvariable or "base" cost and this be the new definition of $T_{o\ell}$. B can now be redefined as κ, the change in cost (or cost consequence) due to a variation in geologic state from that which was assumed. So when the cost of a ground variation from the design state is computed it is defined as the base cost plus the change in cost elements of each given subsystem, or using Equation 4.8:

$$\kappa = \Delta B = \sum_{n=1}^{3} \Delta\xi_n$$

and:

$$T_{\ell a} = T_{o\ell} + \kappa =$$

$$T_\ell + \sum_{j=1}^{4} \sum_{n=1}^{3} \Delta\xi_{nj} \qquad [4.10]$$

Where $T_{\ell a}$ is the cost for a segment of length ℓ subject to a set of ground parameters that can be referred to as "a." This "a" will be shown later to correspond to the identification of the end branch node as shown by the "a" in Figure 1.2.

The change in material cost, $\Delta\xi_1$, is still just equal to the sum of all additional material purchases or equipment modifications so that, as in Equation 4.3:

$$\Delta\xi_1 = \xi_1 = X \qquad [4.11]$$

The labor and operating cost variables, ξ_2 and ξ_3 respectively, can be evaluated together because they are each functions of length and ROA. The change in cost, or $\Delta(\xi_2 + \xi_3)$

for one change in ground state of a given parameter is, from Equations 4.5 and 4.6:

$$\Delta(\xi_2 + \xi_3) =$$

$$\frac{\ell}{ROA + \Delta ROA}(Y + \Delta Y + Z + \Delta Z) - \frac{\ell}{ROA}(Y + Z) \quad [4.12]$$

Where ΔY and ΔZ are the changes in labor cost rate and operating cost rate respectively, and ΔROA is the change in ROA due to a change in ground conditions from those which were assumed. Simplifying Equation 4.12 leads to:

$$\Delta(\xi_2 + \xi_3) =$$

$$\frac{\ell}{ROA + \Delta ROA}[\Delta Y + \Delta Z - \frac{\Delta ROA}{ROA}(Y + Z)] \quad [4.13]$$

Equation 4.9 can now be expressed in terms of cost changes due to the occurrence of a specific set of ground changes (recall that the segment has only parameter states which are mutually exclusive along its length, but that this does not exclude the random occurrence of changes in several of the parameters which constitute the model:)

$$T_{\ell a} =$$

$$T_\ell + \sum_{j=1}^{4}\{X_j + \frac{\ell}{ROA + \Delta ROA}[\Delta Y_j + \Delta Z_j - \frac{\Delta ROA}{ROA}(Y_j + Z_j)]\} \quad [4.14]$$

Where $T_{\ell a}$ is the total cost of the tunnel segment of length ℓ under a set of ground conditions identified as 'a' (as described above) and T_ℓ is the cost of the segment for the original set of ground conditions. The remaining term is no longer the costs of all ground variable elements, B, but is identified as the cost consequence of the occurrence of the "a" set of ground parameters $\equiv \kappa_a$.

Application of this model is made through comparison of $T_{\ell a}$ from Equation 4.14 and K_a of Equation 1.10. (Shown below for a branch with m nodes.) Realizing that the m nodes represent a variation in m possible ground parameters, the nodal cost consequence of a parameter variation, κ_i can be computed from the latter portion of Equation 4.14 or:

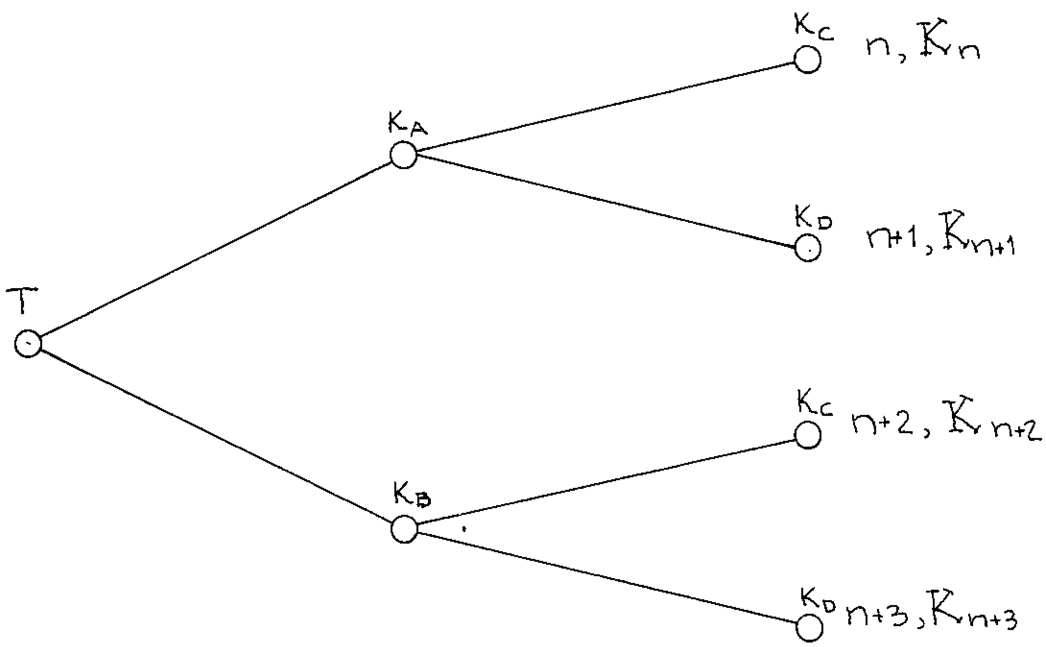
$$\kappa_i = \qquad [4.15]$$

$$\kappa_i = \sum_{j=1}^{4} \{X_{ji} + \frac{\ell}{ROA + \Delta ROA_i}[\Delta Y_{ji} + \Delta Z_{ji} - \frac{\Delta ROAi}{ROA}(Y_{ji} + Z_{ji})]\}$$

Where X_{ji}, ΔY_{ji}, ΔZ_{ji}, and ΔROA_i are all defined in terms of variations due to the occurrence of the parameter state represented by node i. The relationship between κ_i, Equation 4.14, Equation 4.15, and the decision tree model is illustrated in Figure 4.1 for a system where m = 3 and four possible cost consequences κ_A, κ_B, κ_C, and κ_D exist. Summing the cost consequences of each node over an entire branch and adding the base cost, T, the branch cost consequence of Equation 1.10, K_n is derived:

$$K_n = T + \sum_{i=1}^{m-1} \kappa_i \qquad [1.10]$$

It can be seen that K_n, the branch cost consequence, then is equivalent to the cost for the occurrence of the ground con-



$$K_A = \sum_{j=1}^{4} \left\{ X_{jA} + \frac{\ell}{ROA + \Delta ROA_A} \left[\Delta Y_{jA} + \Delta Z_{jA} - \frac{\Delta ROA_A}{ROA} \left(Y_{jA} + Z_{jA} \right) \right] \right\}$$

$$K_B = \sum_{j=1}^{4} \left\{ X_{jB} + \frac{\ell}{ROA + \Delta ROA_B} \left[\Delta Y_{jB} + \Delta Z_{jB} - \frac{\Delta ROA_B}{ROA} \left(Y_{jB} + Z_{jB} \right) \right] \right\}$$

$$K_C = \sum_{j=1}^{4} \left\{ X_{jC} + \frac{\ell}{ROA + \Delta ROA_C} \left[\Delta Y_{jC} + \Delta Z_{jC} - \frac{\Delta ROA_C}{ROA} \left(Y_{jC} + Z_{jC} \right) \right] \right\}$$

$$K_D = \sum_{j=1}^{4} \left\{ X_{jD} + \frac{\ell}{ROA + \Delta ROA_D} \left[\Delta Y_{jD} + \Delta Z_{jD} - \frac{\Delta ROA}{ROA} \left(Y_{jD} + Z_{jD} \right) \right] \right\}$$

$$K_n = T + K_A + K_C = T_{\ell n}$$

$$K_{n+1} = T + K_A + K_D = T_{\ell(n+1)}$$

$$K_{n+2} = T + \cdots$$

$$\vdots$$

Figure 4.1 RELATIONSHIP OF COST MODEL TO A DECISION TREE WITH n END BRANCH NODES ditions represented by branch n, or in fact equal to $T_{\ell n}$, the total cost for a set n of ground conditions.

4.3 FORMULATION OF GEOLOGIC MODEL

The structure of the decision tree geologic model must represent changes in ground conditions which are significant and have a definite cost consequence compatible with analysis using the general cost model of Section 4.2. A list of parameters which are of importance in soft ground tunneling is presented in Table 4.1 and one possible configuration of the mutually exclusive and exhaustive states for each parameter is also shown. There are other possible states for each parameter just as there may be other ground parameters which the engineer may want to include in the tree for investigation. These matters are up to the engineer and depend on his project's needs and the data that is on hand.

The parameters are structured in decision trees as random sets of states; the framework for a model which was described in Chapter 1. Some of the parameters are conditional on others, as in the case of the stability of cohesionless soils which are dependent to some extent on the water inflow. These conditional probabilities are reflected in the decision trees and explained further along in this chapter. For convenience the whole geologic model will be divided into subtrees and identified by the first or "A"

PARAMETER SYMBOL	GROUND PARAMETER	POSSIBLE GROUND STATES
A	Identification	Firm Cohesive } Homo- Cohesionless } geneous Non-homogeneous Mixed Face
B	Gas Occurrence	Yes No
C	Water Inflow	High Low
D	STR	Stable >1 Unstable <1
E	Long Term Structural Adequacy	Yes No
F	Additional Support of Structures and Utilities	Yes No
G	Boulder and Obstacle Occurrence	Yes No

Table 4.1 DECISION TREE PARAMETERS AND CONDITIONS

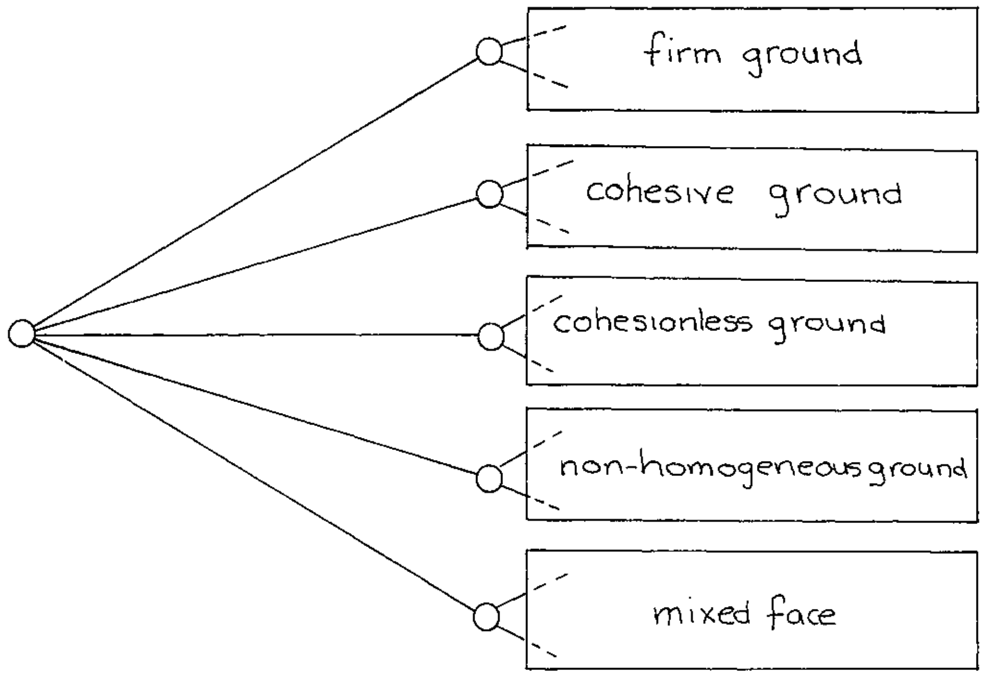


Figure 4.2 GEOLOGIC MODEL SUBTREES

parameter, so that the decision tree may be thought of as illustrated in Figure 4.2.

4.3.1 "A" Parameter Subtrees

The first or "A" parameter is a categorization of the nature of the ground conditions through a segment of tunnel. Segregation of a particular ground into these categories is based on two criteria. The first is dependent on the soil properties of the ground and to what extent they are either granular or cohesive in nature. The second consideration is the dominant features of the area geology which may control the type of construction allowable, and the behavior of the ground.

As long as the tunnel is being constructed in a uniform media, soil properties such as strength and permeability govern the construction characteristics and tunnel fixed costs and rate of advance are affected respectively. In cases where the ground is non-uniform, accurate descriptions of individual soil layer characteristics are relatively of smaller consequence, and the mass characteristics of the layered media must govern the construction. Such layering can be considered as gross changes in material characteristics that are recognizable through normal investigative procedures having an impact on constructions costs. Subtle differences in material properties that either may be measured to some degree, but have no identifiable construction consequence; or conditions that have a significant construction consequence but cannot be predicted with sufficient degree of confidence; must be referred to as a uniform media. Conditions which constitute a discontinuous medium are, for example, permeable layers in an essentially impermeable soil, because the water related properties of the medium are primarily a function of the permeability of the thin, porous layers. Permeability variations on the order of one magnitude more or less occurring altermatively over a space of a few inches, as in a varved clay, however, can be represented by a single mass permeability whose value is equally a function of both soils.

The three categories of the "A" parameter which are soil property dependent are - firm ground, cohesive ground, and cohesionless ground. The remainder of each of the corresponding subtrees is dependent on the specific material properties of a soil showing general behavior which would set it into one of these categories. The difference between firm ground, or Ground I, and Grounds II and III is actually based on an assessment of the unsupported standup time of the soil stratum being tunneled. (The independence of all the "A" parameters from the construction procedure is unique with respect to the other parameters, and this is a major factor in its selection as head of the subtree.) The difference between categories II and III is that of effective grain size and its relation to material behavior as explained in Chapter 2.

The two remaining trees are physical descriptions of geologic stratification. Non-homogeneous soil, Category IV, is entirely dependent on the aggregate behavior of all its layers and therefore can reflect a combination of all the behaviors of Grounds I through III. The last category is really a special case of IV but since rock is not covered in any other category and the problems of advance through the soil/rock interface really mask other factors by diminishing their relative cost significance; the mixed face condition, V, is treated as a separate category.

The segregation of a ground into one or more of these categories is the first step in relating design to ground classification and uncertainty. The identification parameter "A" is more than a nominal step as each classification can have a separate, quantifiable, effect on cost. Different construction procedures may be chosen for a Cohesive, Ground II stratum than for a Cohesionless or Non-Homogeneous medium, because the condition imposed by each of these grounds are different. Therefore, chances of success with a given technique in all five grounds will vary with the type of ground as described by the "A" parameter. Thus the identification parameter is a step on the decision tree rather than just a subtree heading. The remainder of this section is a description of each subtree.

4.3.2 Ground I: The Firm Ground Subtree

A stratum can be characterized as firm ground if it is uniform and of high shear strength. It is everywhere an elastic medium and upon excavation will not behave plastically even if left unsupported for periods longer than days. Elastic behavior can be inferred without recourse to tunneling procedures by examining the stability ratio, Equations 2.4. Firm ground is then defined in terms of ground stresses only:

$$\gamma z/s \leq 1 \qquad [4.14]$$

Where s is the shear strength of the medium.

Ground I is therefore comprised of both cohesive and

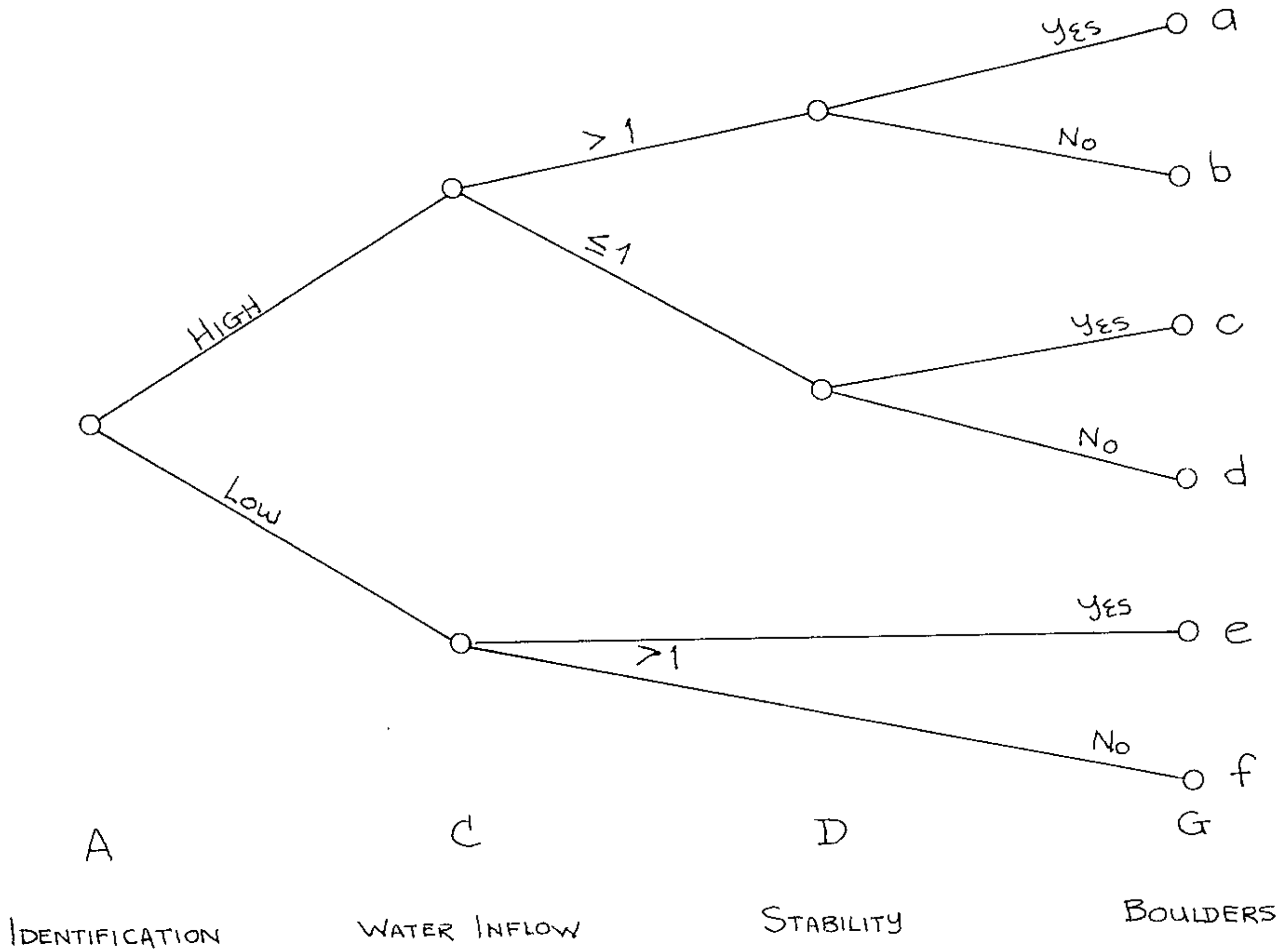


Figure 4.3 FIRM GROUND SUBTREE

cohesionless soils with recourse only to their in situ shear strength. Soils that may fall into this class are stiff, overconsolidated clays, well-cemented sands or gravels, and tills. This category actually, as strength increases, laps over into what might be defined as soft, continuous rock. The entire firm ground subtree is shown in Figure 4.3 and a description of the remaining parameters follows.

a. "C" Parameter, Water Inflow - Water will not flow through firm ground unless the ground is comprised of very porous material, e.g. cemented, coarse sands or gravels. Where the ground is jointed and discontinuous, allowing cracks through which water can infiltrate, the medium must be classified as Ground IV, non-homogeneous, rather than be classified as firm ground. Water inflow can be either high or low, the boundary being assessed in terms of construction consequences rather than having a fixed value. Since in general firm ground does not require stabilization, water inflow is only important to cost as it affects the proposed tunnel dewatering system. The quantity of flow cannot affect tunnel stability enough to require ground stabilization, but can interfere with the construction process and therefore the tunnel dewatering system must be changed to increase capacity.

Assessment of the cost and time spent increasing dewatering can be made, if a value of flow rate or quantity

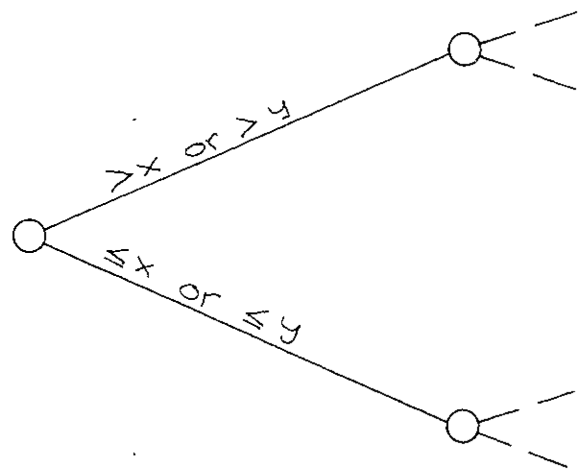


Figure 4.4 FURTHER DEFINITION OF FIRM GROUND "C" PARAMETER

which is the boundary between function of the system and non-function can be assessed. This value can only have meaning with respect to the dewatering system used and therefore must be modified for each trial design. The "C" parameter tree segment would then resemble Figure 4.3 with x in flow, per unit time and y just in flow.

Very specific information is required concerning soil permeability and reservoir storage capacity before such an analysis can be accomplished, so it is very possible the "C" tree segment may have no identifiable cost consequence. The reason then for the existence of this parameter is that, in firm ground, the stability or "D" parameter is affected by water inflow and therefore a resolution of the "C" parameter must precede the evaluation of the construction stability. If water inflow is high, STR (standup time ratio) may be reduced enough to allow local instability, whereas when the state of water inflow is low, the stability will not be affected and therefore will remain STR>1.

b. "D" Parameter, Construction Stability - Firm ground by definition is stable during construction. The only instance of instability takes place when high seepage pressures erode intergranular cementation between uniformly sized soil grains. This behavior is characteristic only of fine to medium, uniform sands, and even so the material cohesion is strong enough overall to be affected only in local, more weakly cemented areas. Very local spalling or dusting which is the failure of a few chips or grains of soil, that does not affect tunneling operations, does not qualify as instability or non-homogeneity.

A local instability, STR<1 for a small segment, can bury an excavator leading to a small delay while it is freed. Such an occurrence may be predicted by an analysis of water related soil properties, the hydrogeologic environment and an assessment of the variation in the strength of particle cementation. If c is judged high enough to withstand predicted pressures for the length of time necessary to install the tunnel liner, t_a, then STR>1 and the soil will be stable for that particular design/construction system. If there is a question that the soil can withstand the pressure over time t_a then may be STR≤1 and the cost and time delay of a local failure should be assessed. Such an analysis requires many subjective judgements by the engineer which leads to uncertainties. Througnout this thesis stability will be regarded as being conditional to water inflow, in fact this is not necessarily true; but used here to demonstrate the principal of conditionality.

c. "G" Parameter, Boulder and Obstacle Occurrence - The occurrence of boulders and obstacles acts to decrease ROA and increase fixed costs in the end. The definition of a boulder is a large soil or rock particle of very high strength, greater than some threshold size in diameter. This threshold diameter is a function of the construction method and tunnel diameter. An obstacle is some non-earth object which is predictable and whose occurrence can affect tunnel construction cost.

Boulders must be broken up into pieces small enough to be within the capacity of the muck system. The severity of their effect on ROA is dependent on the strength of their composition and the frequency of their occurrence. The effects of boulders, or obstacles for that matter, on firm ground excavation are somewhat diminished by the fact that the construction methods employed are generally fairly open at the face and therefore facilitate access to the face for breakup of the rock. The degree of cementation between boulder and soil also has an effect on the ROA since the more soil/boulder cementation that exists, the more difficulty is to be experienced in boulder removal (until of course cementation becomes so great that the material can be treated as a rock and a tunnel boring machine can be used.)

Obstacles sometimes can not be predicted, as in the case of the BART tunnel under the Ferry Building (32) where old timber piles were encountered which slowed excavation and cost $750 each to be removed. Such obstacles, however, cannot be included in the probabilistic approach by the very nature of their being "unknown" which automatically fixes the probability of their occurrence as nil. If however the presence of such piles could be inferred by the examination of old plans or reports, without the condition or location exactly specified, then some quantifiable uncertainty which could be narrowed, or be better defined by further investigation, is possible. A long abandoned sewer, for instance, might somehow be known to exist within the area of the project without the exact location being known. Then the cost of further investigation versus the decision-making advantage of increased certainty could be assessed and a decision concerning further investigation could be made. Table 4.4 shows the general procedure for evaluating the need for in-

1. Estimate Nodal Cost Consequences
2. Assign Probabilities of Occurrence of Each State
3. Compute Sub-tree EMV
4. Estimate Extremes of State Probabilities Due to Site Investigation
5. Re-evaluate EMV for Both Extremes
6. Make Decision

Table 4.4 PROCEDURE FOR ASSESSING NEED FOR FURTHER SITE INVESTIGATION

vestigation of the existence of obstacles using Baye's Theorem in the fourth step. The need is wholly dependent on the cost consequences and the ability of the investigation to resolve the uncertainties.

4.3.3 Ground II: The Cohesive Ground Subtree

The distinguishing characteristic of the cohesive subtree is a tendency to plastic material behavior which is characterized by large deformations before material failure. It is distinguishable from Ground I by its stability ratio which in general is:

$$\gamma z/s > 2 \qquad [4.15]$$

and s is the undrained strength s_u. The Tunnelman's Classification categories of very soft squeezing, swelling, squeezing, and fast and slow raveling fit as a behavioristic description of Ground II. The cohesive ground definition would not allow a material classified as firm ground unless

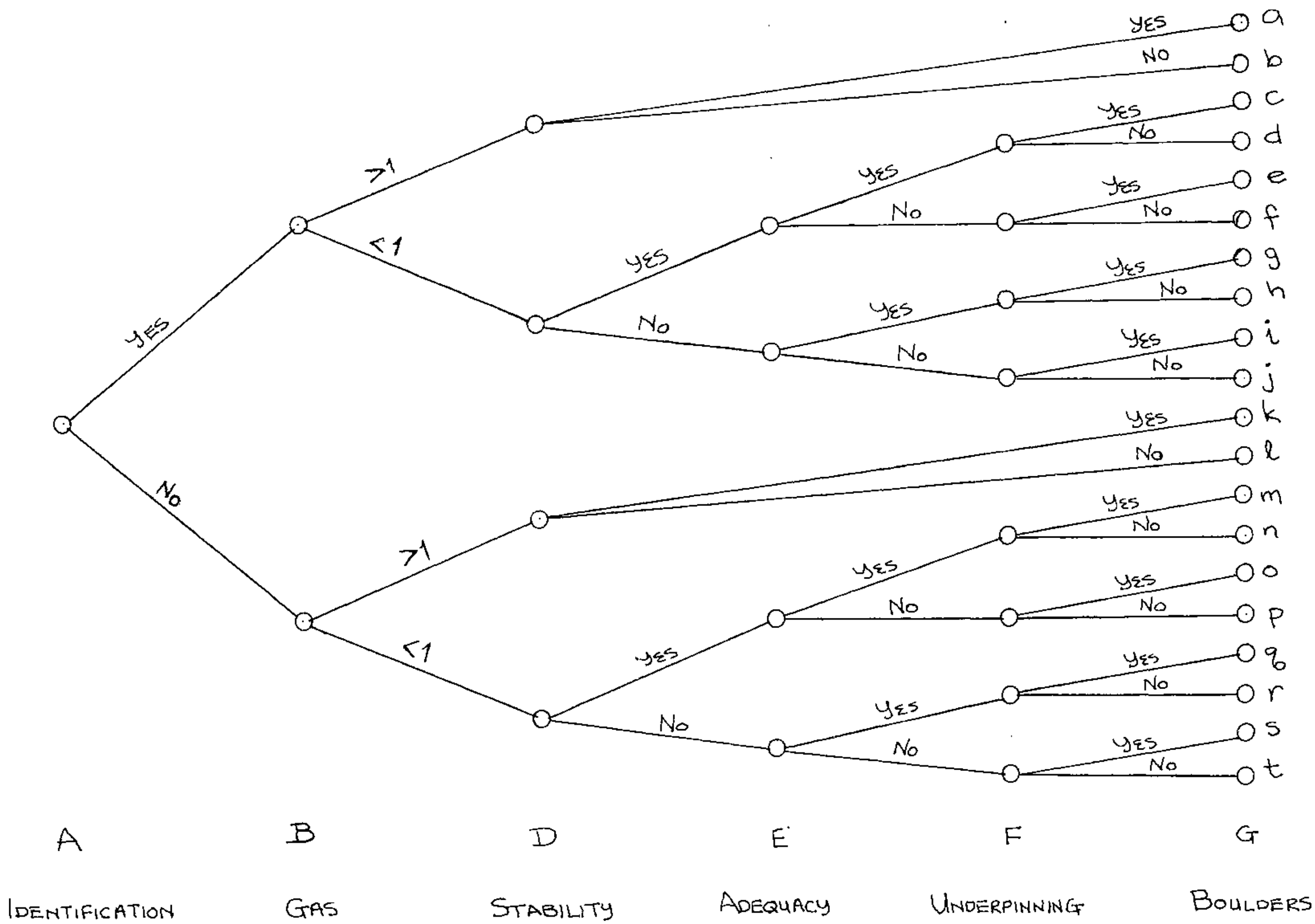


Figure 4.5 COHESIVE GROUND SUBTREE

this state is achieved through ground stabilization. Cohesive ground is impermeable to gas and water although gas can exist in organic man-made fill. The full cohesive subtree is shown in Figure 4.5,

a. "B" Parameter, Gas Occurrence - Cohesive soils are of very low permeability and therefore not usually infiltrated by gas. In man-made fills, however, it is possible for all the air voids to be saturated with gas which originated from the decay of organic matter.

b. "D" Parameter, Stability During Construction - Very important in the construction of a tunnel through cohesive ground is the stability ratio which includes the internal pressure supplied by the ground stabilization as in Equation 2.4. It is the inclusion of the construction procedures which separate the stability ratio of the "D" parameter from those of the "A" where the construction procedure is ignored. The question posed with respect to stability in this parameter is, "Will the support provided the ground be timely and sufficient?"

As construction proceeds in a saturated normally consolidated clay, air pressure may be used as the internal pressure p_i to reduce the pressure of the ground p_z ($=\gamma z$.) That pressure may or may not be sufficient to decrease the total soil pressure enough to make SR <6 which is the criteria for stability. So for determination of the value of

STR which describes the stability, the stability ratio may be used:

$$STR>1 \qquad (\gamma z - p_i)/s_u < 4 \qquad [4.16a]$$

$$STR \approx 1 \qquad 4<(\gamma z - p_i)/s_u < 6 \qquad [4.16b]$$

$$STR<1 \qquad (\gamma z - p_i)/s_u \geq 6 \qquad [4.16c]$$

The above equations apply to stability of a vertical face and each has a definite consequence with respect to construction. If STR>1 there will be no problem, the design/ construction scheme applies for the given soil condition. As STR→1, stability becomes a problem, the ROA slows to ensure the safety of the workmen and the prevention of a collapse. Fixed costs are not affected, but variable costs may be as more labor goes into the construction per unit length of tunnel. STR<1 indicates that the tunnel cannot be constructed without a change to the design system. Such a change is necessitated by either unworkable conditions or a disturbance to the environment which is unacceptable. Cost for modification of the system is reflected as an increase in fixed costs, a lump sum addition of time due to delay which increases ROA, a change in the average construction cycle time due to the modification of the construction cycle, and a penalty which could be representative of premiums which might have to be paid for materials, contract penalties, or legal fees.

Another method of attacking the identification of overall stability is through an estimate of tunnel standup time versus application of load as mentioned in Chapter 3. This would be applicable for stiffer soils whose deformation with time can be estimated, or where the elastic range is finite in length of time. Such a soil would be described in the Tunnelman's Classification as slow or fast raveling. The application time of support can be estimated from the liner erection time and/or the projected ROA of the excavation cycle. The actual criteria for decision then would appear as in Figure 4.6 or be selected as decided by the engineer. He may decide that two branches, stable and unstable, are sufficient to divide the cost consequences of tunnel instability, this is his option. To simplify the presentation only two alternatives for stability are shown on the decision trees presented.

c. <u>"E" Parameter Selection, Long Term Structural Adequacy</u> - The adequacy of the tunnel support system over long periods is of dubious certainty only in very soft, sensitive, or swelling clays. These soils are readily identifiable by the proper laboratory tests which may be conducted on material from the layer when encountered.

Soft soils are recognizable from their stress history, chemical analyses for organic materials, and Atterberg Limits. Their undrained strength will always be so characteristically low that STR>1 will never be achievable due to

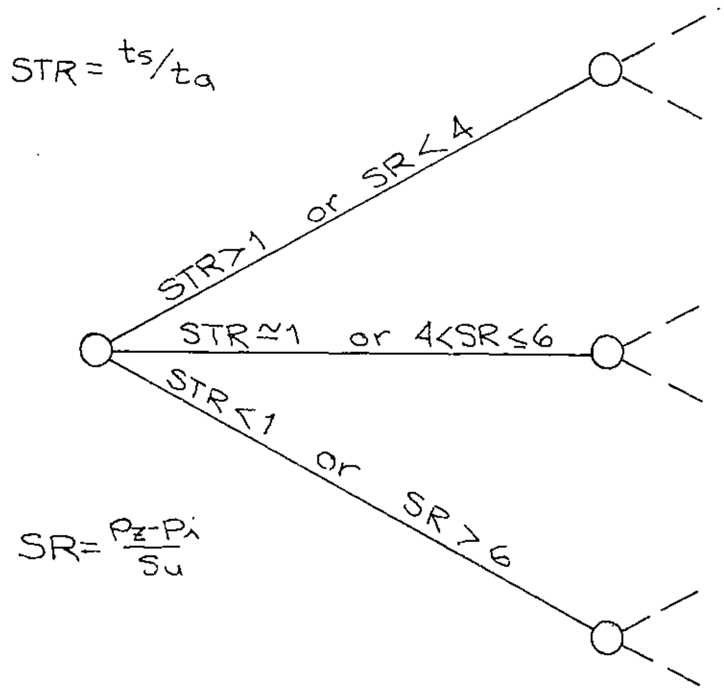


Figure 4.6 CRITERIA FOR SEGREGATION OF PARAMETER "D"

very low t_s. Therefore parameter "E" is conditional upon $STR \leq 1$.

Sensitive clays can be identified through consolidation tests and are identified by very steep, sometimes vertical, slopes on the virgin compression portion of e-log p curves. Swelling clays are frequently characterized by very high PI's and w_L, and have a large affinity for water.

d. <u>"F" Parameter, Additional Support of Structures and Utilities</u> - A decision can be made either in favor or against protection of structures along the route of a tunnel construction. If all structures are underpinned or otherwise supported through a given segment, then this parameter has little meaning and the state is registered as no. Since underpinning all structures is both uneconomical and usually unnecessary, however, the maximum protection is hardly ever applied in real life and consequently some amount of damage or the need for additional protection may be possible. Decisions concerning underpinning are usually made at an early stage and are based many times more on the relative importance of the structure than on analytical assessments of the effect of tunneling on the movements of the structure. Soil movements are however difficult to predict and it is also difficult to assess what effect soil movement will have on structural damage.

Surface subsidence is related to the amount of ground lost during construction and encroachment of soil into the annular void as explained in Chapter 3. Other problems can arise when a compressible layer is located between construction and the tunnel. Both occurrences depend on the amount of disturbance caused by the construction which in effect makes this parameter conditional on the stability during construction.

e. "G" Parameter, Boulder and Obstacle Occurrence - The same definition of boulders and obstacles and their consequence applies here as appeared in Section 4.3.2.c. The occurrence of these events may in fact be more serious in plastic soils where stabilization of the face will tend to hinder accessibility to the face. Where the face is closed off to access, a boulder of smaller diameter occurring at widely spaced intervals will have the same effect as larger, more frequent boulders in more open construction. In verv soft soils where construction must be carefully controlled, the occurrence of even a few boulders of relatively small size can have very negative effects on the construction and large additional costs and delays can be anticipated. Boulders may be anticipated, as indicated in Chapter 2, in glacial deposits and residual soils close to bedrock.

4.3.4 Category III:The Cohesionless Ground Subtree

The cohesionless ground subtree is perhaps a slight misnomer in that the material involved in Category III may display apparent, and even some slight true cohesion. It can be distinguished from Ground I, however, by the yielding zone about the excavation which has been described by Schmidt et al (32) as developing where:

$$(1 - \sin\phi)/\cos\phi < (p_z - p_i)/c \qquad [4.17]$$

In terms of the Tunnelman's Classification these soils are classified as fast and slow raveling, running, and flowing. Cohesionless ground can be distinguished from cohesive or Ground II by its much greater permeability and is comprised of soil particle sized from permeable silts to gravels. Figure 4.7 shows the entire subtree and its end node consequences a to t.

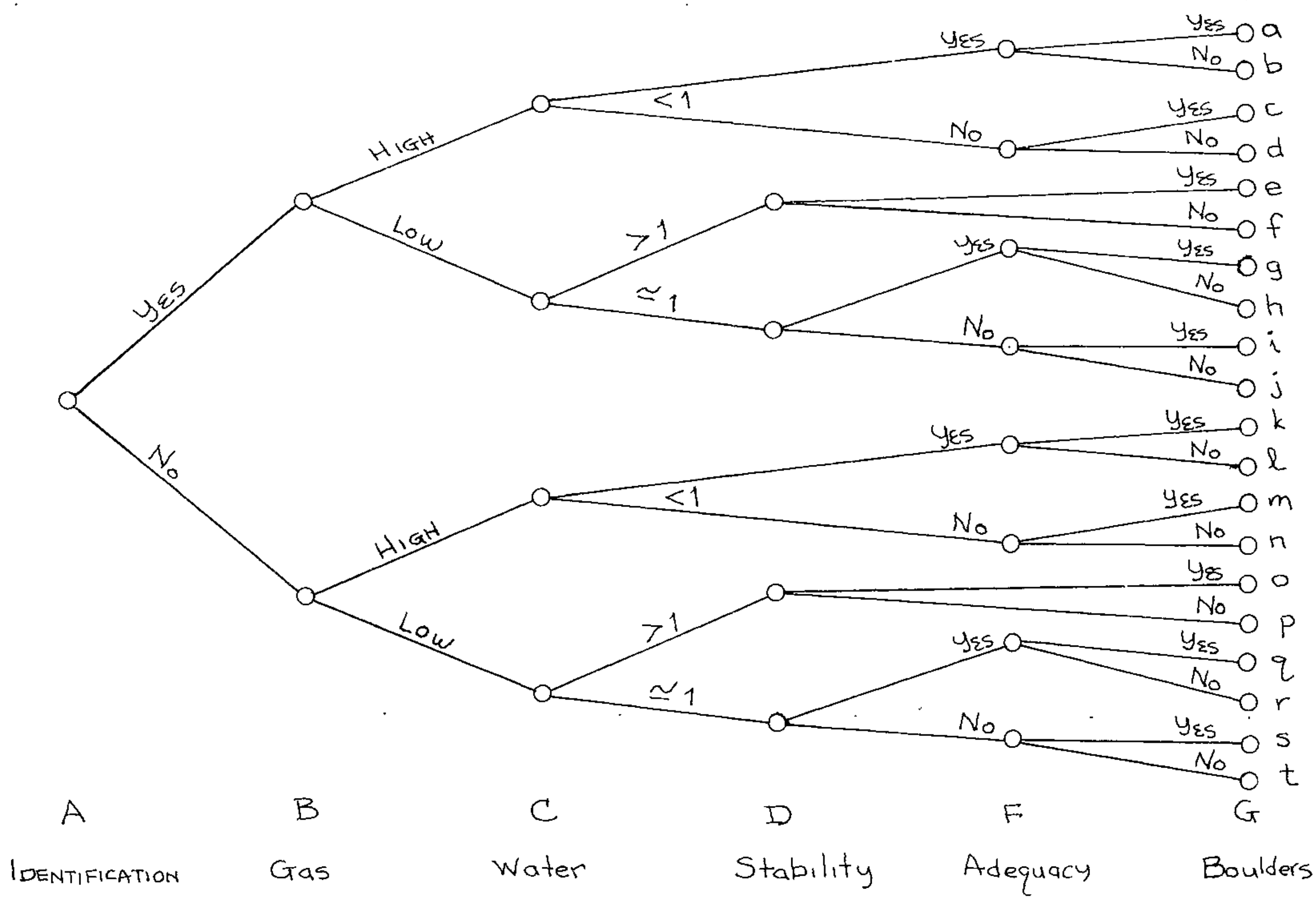


Figure 4.7 COHESIONLESS GROUND SUBTREE

a. <u>"B" Parameter, Gas Occurrence</u> - Gas may be a problem in permeable soils, because it can collect beneath impermeable strata and flow into an excavation which penetrates the gas bearing strata. Advance warning of such a stratum, which must occur or catastrophic results may be a consequence, will necessitate an increase in ventilation costs and possibly an increased cost due to the purchase of new equipment or modification of the existing equipment.

b. <u>"C" Parameter, Water Inflow</u> - The occurrence of water in essentially cohesionless soils can have potentially dangerous consequences. As in Ground I, one of its most critical aspects is the effect on construction stability, although here the penalties are much greater if high inflows are allowed to occur.

A high inflow in a material where s is very small can result in either a run, which is a temporary delay, or in a flow which could close large segments of the tunnel and even force indefinite halt to the construction. The occurrence of such events is dependent on the source of the water, the storage capacity of the reservoir, the permeability of the cohesionless media and the connector to the water supply, and the type of dewatering system used. Usually if the possibility of water inflow is recognized in advance, the water inflow problems are slight and depend only on minor features of the geology such as the occurrence of perched water. The relative density of the medium is very important in determining the quantity of water which will cause trouble with stability.

c. "D" Parameter, Overall Stability - Concern for overall stability in a cohesionless medium is dependent on two factors, water inflow and relative density. Water inflow will always cause instability when it is high, but can be counteracted somewhat by either compressed air or grouting.

Compressed air is less effective as the D_{10} of the soil increases, but the effectiveness of grouting increases.

d. "F" Parameter, Additional Support of Stuctures and Utilities - The need to add additional support for structures and utilities will arise only where instabilities occur which cause yielding up to the level of structure foundations or into compressible soils. The discussion which was presented in Section 4.3.3.c applies here with the addendum that this is usually not a problem in Ground III except in material of low relative density.

e. "G" Parameter, Boulder and Obstacle Occurrence - Again the discussion of Sections 4.3.2.c and 4.3.3.d are pertinent even though access to the face is usually more viable in cohesionless soils.

4.3.5 Ground IV: Non-Homogeneous Ground

The occurrence of non-homogeneity is possible in almost any soil. It can manifest itself as a discontinuity such as a failure plane or fissures in an otherwise homogeneous medium; or it can be the occurrence of many layers and pockets of materials of varying properties. The result is the same in any case, the behavior of the soil is now governed by the macroscopic rather than microscopic characteristics of the material and the prediction of the behavior must rely now more on field and related tests rather than laboratory tests or the identification of the specific materials. The identity of a category IV medium then can have two meanings. The occurrence in itself of variable ground which is the presence of one or more boundaries of significantly differing materials in the same cross-section; or the appearance of discontinuities in the properties of an otherwise uniform material.

The number of boundaries which can occur in a given cross-section has a physical limit before the soil's behavior can be described as homogeneous and either cohesive or cohesionless. Such a soil may be glacial in origin, as a till, where no amount of investigating is going to expose all the soil boundaries. Major discontinuities in so-called homogeneous layers are only important if they affect either the permeability or the strength of the material. Minute fissures for instance, though discontinuities, in a stiff clay will have no significance with respect to permeability since they usually will not be connected to any water of significance. As for strength, the amount that standup time is affected by the fissures is insignificant for the short period of construction depending upon the construction procedure. Figure 4.8 depicts the Category IV subtree and its consequences a to x.

a. "B" Parameter, Gas Occurrence - Gas is most likely to occur in layered soils which once contained organic materials and where impermeable layers overlie permeable ones. Fills can also sometimes be termed as non-homogeneous and contain gas.

b. "C" Parameter, Water Inflow - There are three possible states of water inflow which could occur. High continuous inflow when the layer is connected to an undiminishable source, high temporary flow when the water source is limited and low or no flow. Both high inflows will cause instability with the effect on construction increasing as the quantity of water increases.

The most common occurrence of the effects of this parameter will be with lenses of water bearing, permeable materials in low permeability cohesive materials. The occurrence of such lenses will cause delays and increased costs due to dewatering or chemical stabilization ahead of the face. Most of the time the water in the lenses will flow into the construction, hampering work and causing instabilities.

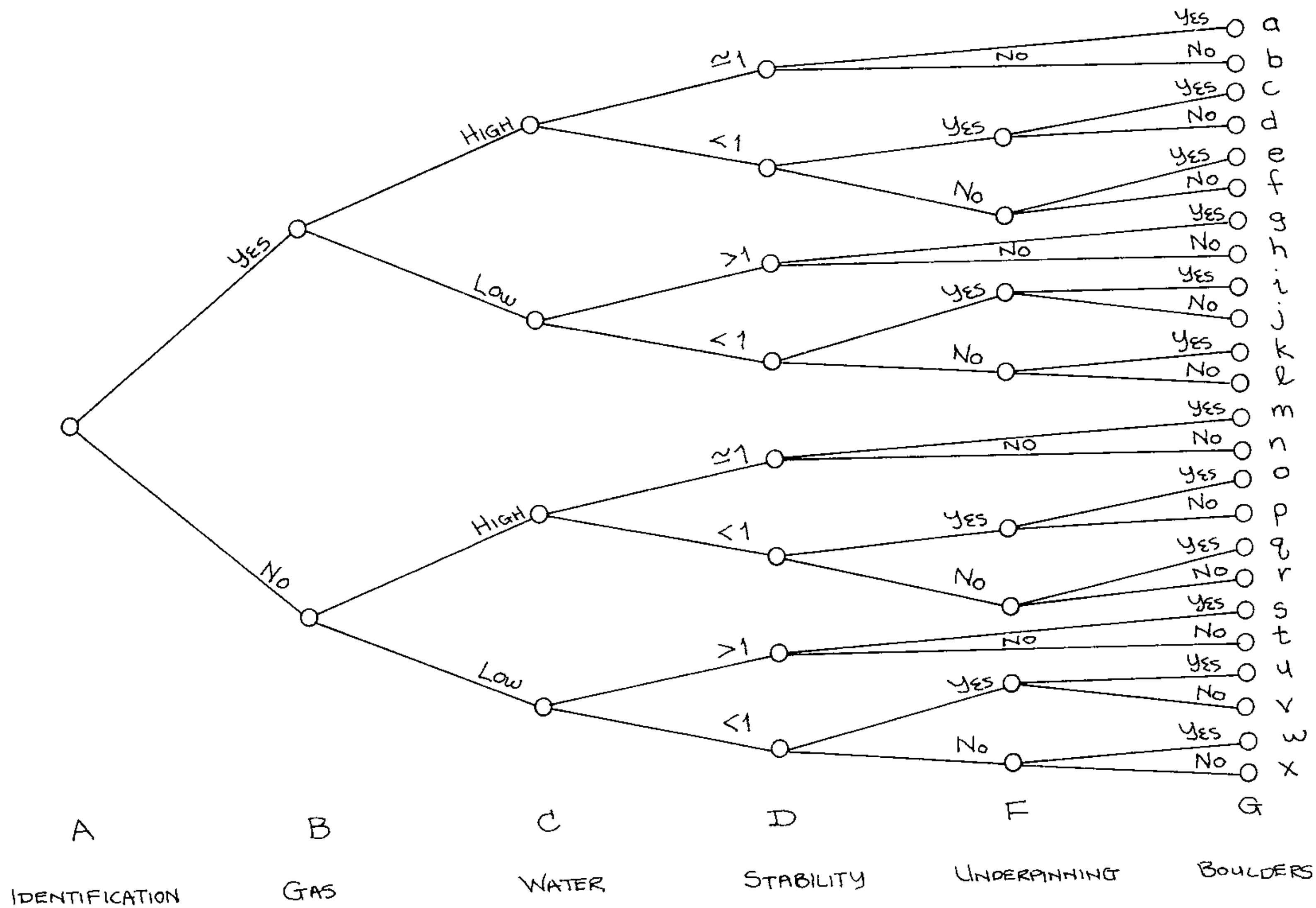


Figure 4.8 NON-HOMOGENEOUS GROUND SUBTREE

c. "D" Parameter, Overall Stability - The feasibility of stabilizing these soils many times is very slight because of the great variations in the material, especially in its permeability. It is highly probable then that areas which remained unpenetrated by grout or lenses which remained water bearing due to their impermeable connections to the surrounding soil will result in local instabilities, STR≈1, and cause delays. Dry, cohesionless soils in pockets of very low relative density result in the same type of instabilities.

d. Other Parameters - Because of the possible dual nature of the non-homogeneous soil, i.e. it can be cohesive and cohesionless in the same cross-section, it is subject to an aggregation of the problems discussed in the parameters in Sections 4.3.2 through 4.3.4, and the same general characteristics with respect to boulders and additional underpinning apply to non-homogeneous soils as apply to cohesive, cohesionless, and firm soils depending on which category of material the non-homogeneous portion resembles. It should be understood however that the ROA in Ground IV will be slower than in the first three grounds described under similar parameter states simply because of the increased possibility in non-uniform soils of encountering adversity, therefore more caution is excercized during construction.

4.3.6 Ground V: Mixed Face

The occurrence of a rock interface in the cross-section of the tunnel excavation has a great impact on the fixed and variable costs of tunnel construction. Excavation has a dual nature and often two different systems must be used to handle both the soil and the rock. The stability of the soil is affected because of the slowed ROA which translates to a longer time the soft ground remains unsupported. It is also very difficult to dewater fully to the rock surface, so water inflow may be difficult and expensive to control. Boulder occurrence just above the rock interface is also a very definite possibility which must be taken into consideration. Other parameters in mixed face construction are similar in nature to those described in Grounds I through IV depending upon which type of ground interfaces with the rocks and therefore no further descriptions of parameter states are related here. The whole subtree is shown in Figure 4.9.

4.4 Application of Probabilistic Approach

The tunnel design has been modeled with respect to both cost and geologic uncertainty and the ground conditions have been probabilistically structured in the decision tree framework. The models of tunnel and ground are the first two steps of the probabilistic design cycle as described in Figure 1.1. The remainder of the process is to allow the two models to interact, checking their adequacy in relation to

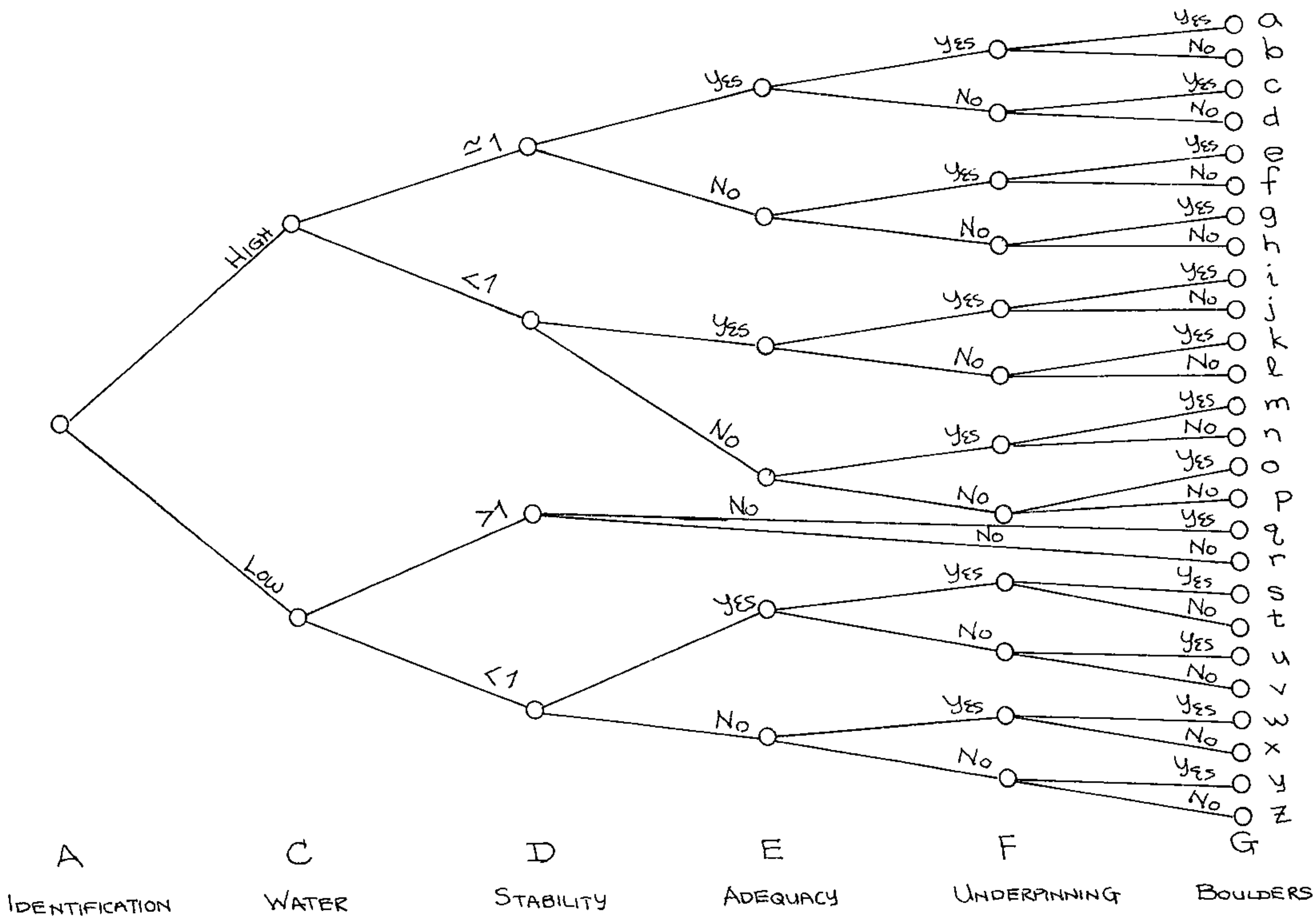


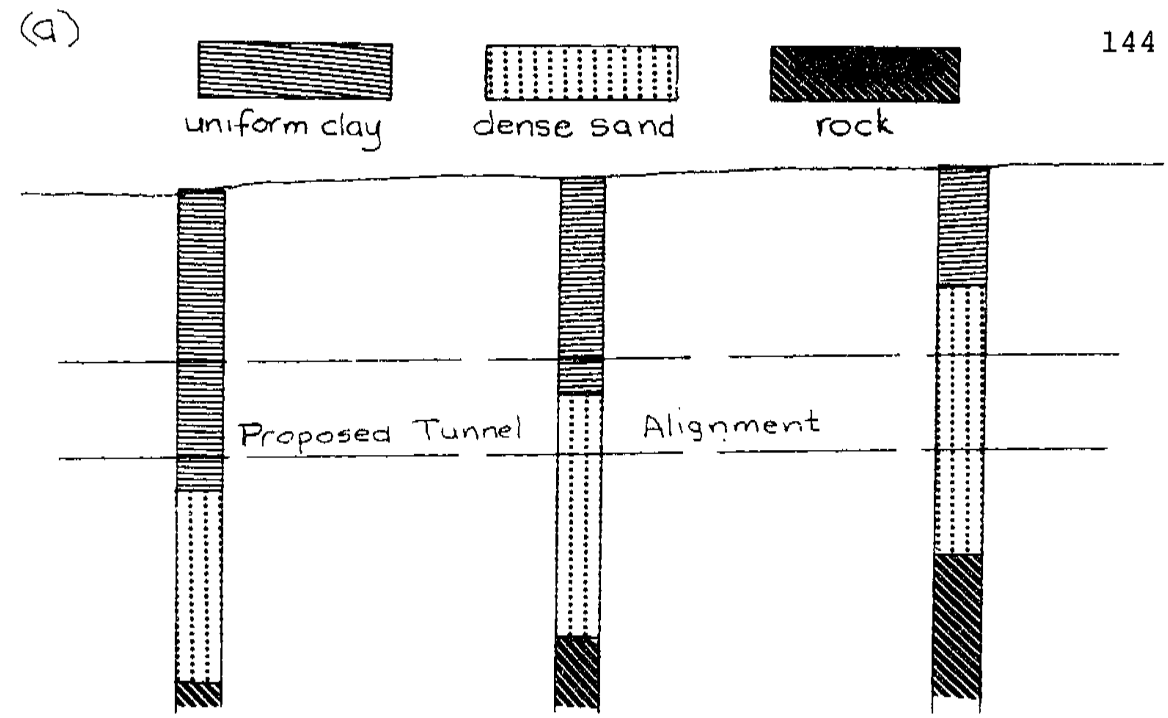
Figure 4.9 MIXED-FACE SUBTREE

the specific needs of the design, and finally to make the decision. With the probabilistic and decision-making tools that have been described, one cycle of the design would consist of the following steps:

1. Separate project into segments in which only one combination of geologic parameter states can exist.
2. Select design alternatives for a single set of soil parameters.
3. Model ground conditions selecting probabilities for each possible alternative state of each parameter.
4. Estimate cost consequences of occurrence of each alternate state.
5. Compute EMV costs for each end branch node.
6. Compute EMV for whole tree.
7. Compare with EMV's for all design alternatives.
8. Make decision.

4.4.1 Converting Geologic Evidence into Model Framework

Figure 4.10a shows three boreholes along a hypothetical tunnel alignment of length $\overline{EF}$ and the evidence of three uniform materials encountered in the boreholes shown. Conventional interpretation of this soils data could result in a soils profile which could be as shown in Figure 4.10b. The



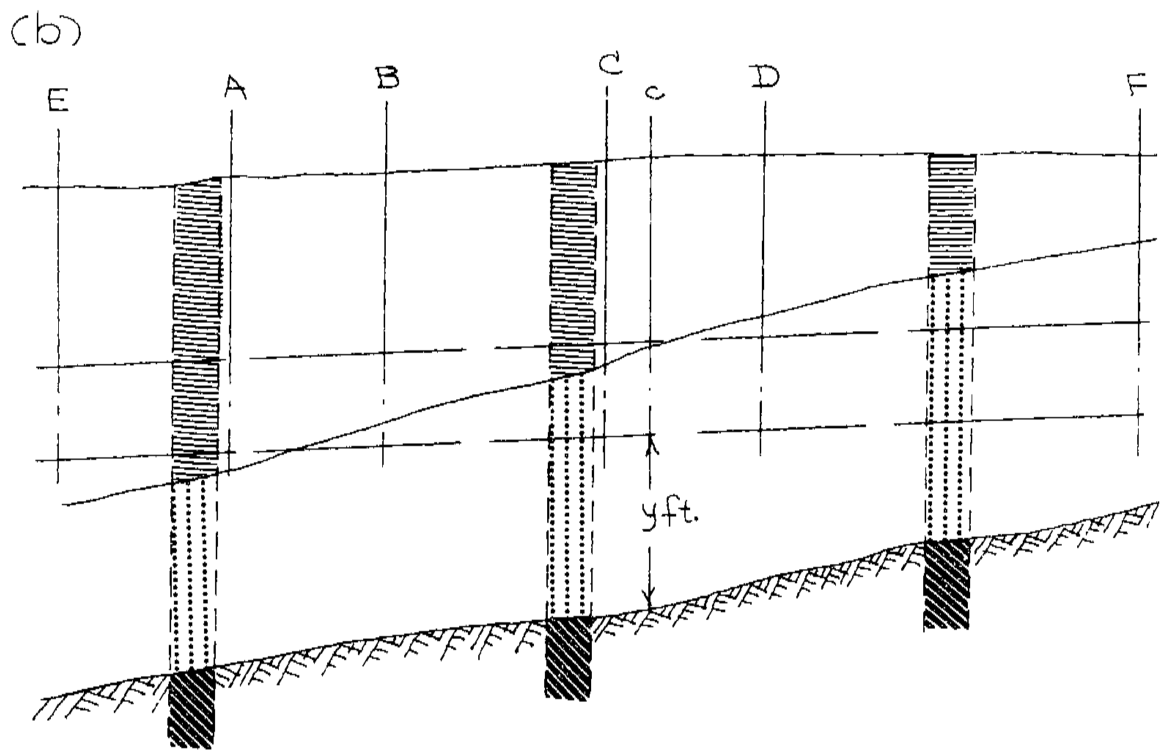


Figure 4.10 TRANSLATION OF BOREHOLE DATA TO GEOLOGIC MODEL

alignment is now divided into segments according to the state of the "A" parameters which may be encountered in each segment and probabilities are assigned. A breakdown of "A" parameter trees per segment with assigned probabilities may be as follows:

Segment	P[Ground Category]
EA	P[II] = 1
AB	P[II] = 0.5, P[IV] = 0.5
BC	P[IV] = 1
CD	P[IV] = 0.5, P[III] = 0.5
DF	P[III] = 1

Other assessments of the geology may result in different segmentation and/or assessment of the uncertainties, this is a function of the amount of information available concerning the geologic strata and the engineer/geologist who is performing the analysis and his experience with the local geology.

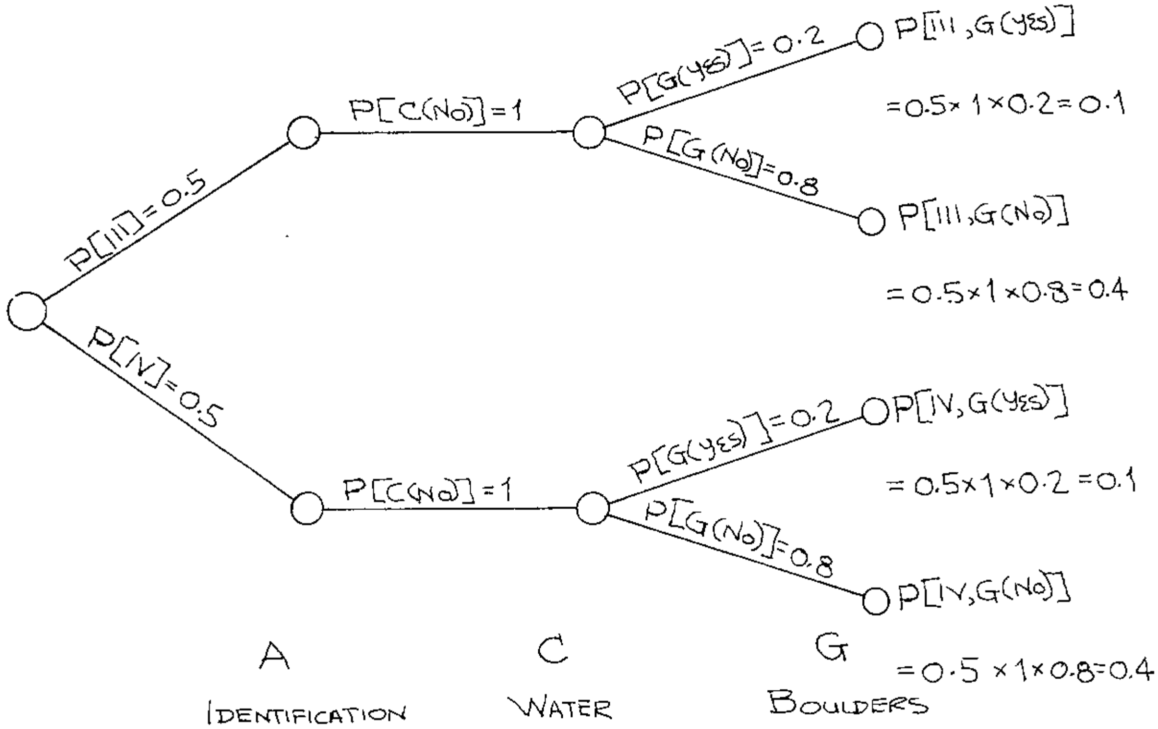
The next step is a careful assessment of the uncertainty of all the other parameters occurring throughout each segment. Further segment subdivision is accomplished to reflect the possible variation of the remainder of the parameter states throughout the tunnel length. If the designer, for instance, estimates from experience with local geology that there is a 20% chance of encountering boulders (the "G" Parameter) when the tunnel invert is within y feet of the bedrock surface; then segment CD must be further divided into two segments. Figure 4.10 illustrates the division of CD into Cc and cD neglecting the uncertainty of locating the surface of the bedrock (this can be taken into account by further segmentation of the alignment, but is neglected here just to simplify the example.) The geology for segment cD, assuming no chance of gas occurrence (P[B(yes)] = 0,) can be modeled probabilistically as shown in Figure 4.11.

4.4.2 Selection of Construction Strategies and Further Assessment of Geologic Model

The remainder of the geologic model cannot be evaluated until a construction system is formulated. For this particular case, since a variety of soil conditions are crossed by the project, a system which can operate successfully in a diversity of geologic settings is a necessity. The choice may be then between a system utilizing hand excavation methods or one using a machine excavator which depends on successful stabilization to attain a maximum rate of advance. The two choices may be formulated as shown:

System	A	B
Components	Machine Exc. w/ Shield	Hand Exc. w/ Shield
	Predrainage	Predrainage
	Liner in Shield	Liner in Shield
	Muck Conveyor	Muck Conveyor

The cost for each subsystem is estimated assuming a particular state of each geologic parameter. Cost consequences are



cD P[III, G(No)] = 0.4 P[III, G(Yes)] = 0.1

P[IV, G(No)] = 0.4 P[IV, G(Yes)] = 0.1

Figure 4.11 ASSESSMENT OF UNCERTAINTY OF CONSTRUCTION INDEPENDENT STATES OF SEGMENT cD then estimated for the occurrence of each possible ground state for each alternate design and end branch node EMV costs are calculated.

For predrainage, for instance, on the upper portion of a permeable layer (unconnected to any large water supply) one could expect that the probability of the success of the dewatering effort is close to unity. Since the layers of sand are dense, it can be expected that as long as the excavation is in the sand alone, there will be small chance of instability. Excavation along the interface will be known to be at least potentially uncertain with respect to stability. Considering all factors then a reassessment of the tree might look as shown in Figure 4.12.

The cost of each path through the decision tree must be formulated by summing the cost consequence of each node. An assessment for end branch node costs for both systems may then be as follows:

End Branch Node Cost	System A	System B
K_a	\$1,000,000	\$900,000
K_b	600,000	800,000
K_c	1,100,000	950,000
K_d	650,000	825,000
K_e	1,200,000	975,000
K_f	850,000	850,000

The above costs are purely hypothetical and were not

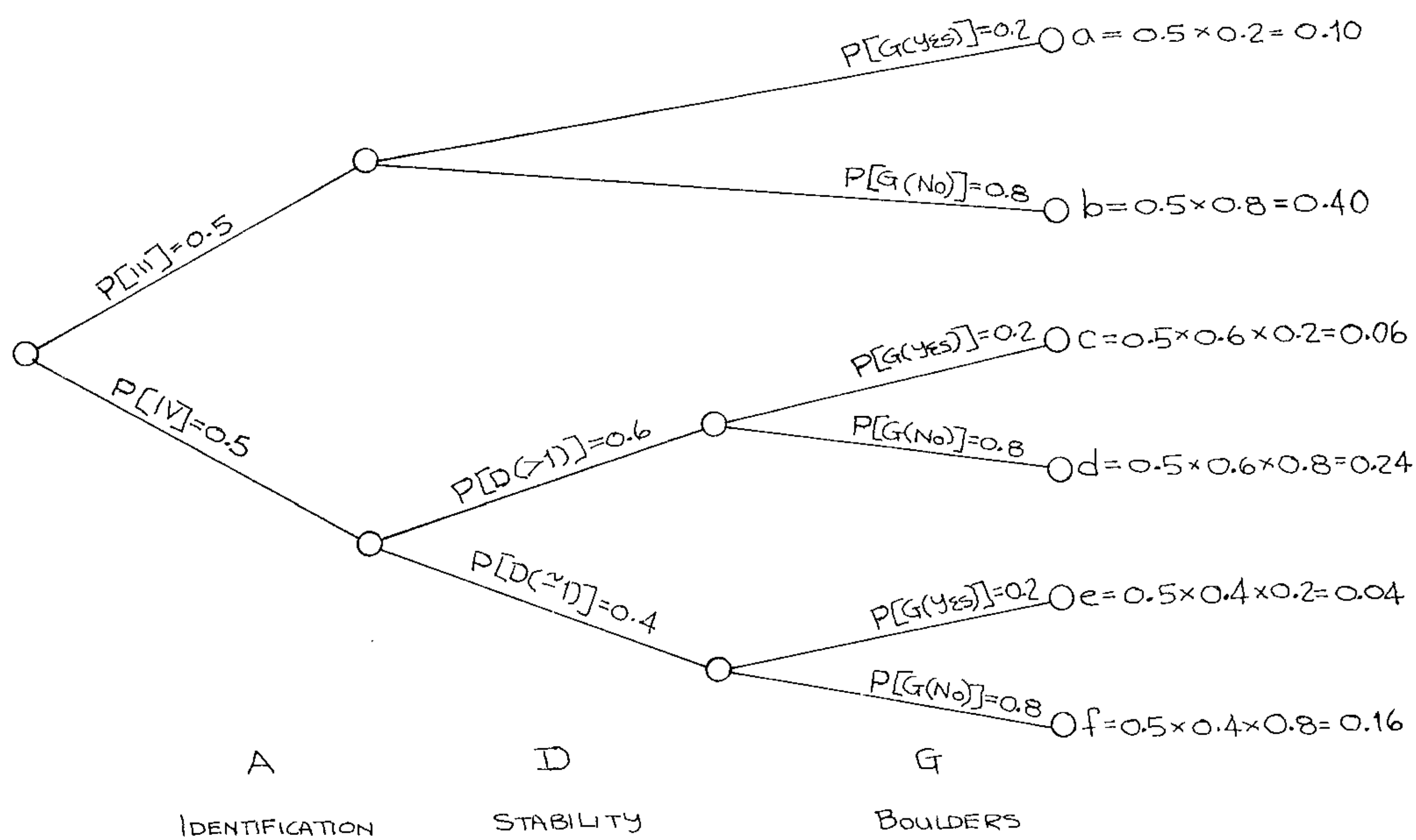


Figure 4.12 ASSIGNING UNCERTAINTIES TO THE cD SEGMENT DECISION TREE

calculated, but merely assumed as an example to help illustrate this design system. If one were actually to estimate these end branch cost consequences for segment cD, then the logical start would be to assume a given set of states for each parameter shown in Figure 4.12. One could assume for example that the boundary between the uniform clay and dense sand will be encountered, therefore identifying this as a Ground IV material; but that the material would be stable for each construction procedure,and that boulders would not be encountered. This set of states for the geologic parameters correspond to the branch of the decision tree with end branch node probability d. The cost consequence corresponding to end branch node d, K_d, is just the first part of Equation 4.14, or T_ℓ. For construction system A, K_d = \$650, 000; while for construction system B, K_d = \$825,000. As mentioned before these numbers have been assumed for this example; but in real practise would be derived from data in the engineer's possession for the geologic parameter states of branch "d."

To compute the end branch node cost consequence corresponding to end branch node probability a, i.e. K_a, one would utilize the model represented by Equations 4.14 and 4.15 and calculate the nodal cost consequences for each parameter along branch "a." To compute the nodal cost consequences from Equation 4.15:

$$\kappa_i =$$

$$\sum_{j=1}^{4} \{X_{ji} + \frac{\ell}{ROA+\Delta ROA_i}[\Delta Y_{ji} + \Delta Z_{ji} - \frac{\Delta ROA_i}{ROA}(Y_{ji} + Z_{ji})]\} \quad [4.15]$$

The engineer must estimate the change in material cost, X_j, labor cost rate, ΔY_j, operating cost rate, ΔZ_j, and rate of advance, ΔROA, for each of the i nodes. In branch a of the decision tree in Figure 4.12 there are only two nodes; one with a cost consequence, κ_1, the difference in cost due to tunneling in a dense sand instead of the medium with the sand/ clay interface, and the other with a cost consequence, κ_2, the additional cost due to encountering boulders along the alignment.

Following this example through qualitatively; the engineer would next assess the cost consequences for the machine excavator, system A, and the hand mining, system B. Tunneling by machine in the uniform sand layer should lead to an increase in the ROA which would lead to a negative cost consequence. For purposes of the example this cost consequence will be estimated (not calculated) as $\kappa_1(A) = -\$50,000$. For the handmining operation, it can be assumed that ROA will also increase (ΔROA negative) but not as much as for the machine excavator, therefore $\kappa_1(B) = -\$25,000$. The occurrence of boulders will probably impact the machine excavator tunnel design more than the hand mined design, so for the positive occurrence of this parameter $\kappa_2(A) = +\$400,$

000 and $K_2(B) = +\$100,000$ respectively, are chosen.

The end branch cost consequences for each system can then be computed by summing each nodal cost consequence or:

$$K_a(A) = \$650,000 - \$50,000 + \$400,000 = \$1,000,000$$

$$K_a(B) = \$825,000 - \$25,000 + \$100,000 = \$900,000$$

The values chosen for each nodal cost may be argued, they were only chosen to indicate trends of costs and to illustrate the application of the model to the decision tree framework. The remainder of the end branch cost consequences can be calculated in a similar fashion.

The computation of the EMV for Segment cD by each construction system is the sum of the end branch node costs, or

$$(EMV)_{cD} = 0.1K_a + 0.4K_b + 0.06K_c + 0.24K_d + 0.04K_e + 0.16K_f$$

For system A:

$$(EMV)A_{cD} = 0.1(\$1,000,000) + 0.4(\$600,000) + 0.06(\$1,100,000) + 0.24(\$650,000) + 0.04(\$1,200,000) + 0.16(\$850,000) =$$

$$(EMV)A_{cD} = \$746,000$$

For system B:

$$(EMV)B_{cD} = 0.1(\$900,000) + 0.4(\$800,000) + 0.06(\$950,000) + 0.24(\$825,000) + 0.04(\$975,000) + 0.16(\$850,000) =$$

$$(EMV)B_{cD} = \$840,000$$

Comparing the results of EMV for Segment cD for both systems A and B shows that system A fits the uncertainty of the ground better than B and therefore A may be a more viable alternative for this one segment - given the accuracy of the evaluation of uncertainty. That is, system A is more suited to the projected ground conditions of section cD than system B. The next step is to calculate EMV over the whole length of the tunnel project which is a sum of all segment EMV's for each construction system. Comparing total EMV for each system over the length of the tunnel will yield a final ranking of the designs based on the probabilistic distribution of ground conditions over the system.

This example has been purposefully simplistic, meant only to demonstrate some of the more important features of the probabilistic approach. For each construction system the probability of the occurrence of each geologic parameter state was assumed to be the same. This would not necessarily be so in a real case. The stability ("D") parameter especially is likely to vary depending upon the construction system. Also many more states and parameters are likely to be analyzed and assessed for each segment, and for a given project many more segments than are shown in Figure 4.10 are likely to be required. Consequently the use of a computer to facilitate computation of project EMV's would probably be a timesaving and economical step.

4.5 SOME FINAL COMMENTS ON THE PROBABILISTIC APPROACH

The approach presented in this paper, using equivalent monetary values (EMV) and the decision tree concept is only an example of one vehicle which may be employed to include the assessment of geologic uncertainty in the design and construction of soft ground tunnels. The utilization of this approach assumes that the effect of geologic uncertainty is significant and can affect design decision making. One set of parameter states, or one branch on the decision tree as shown in Figure 4.13, will be the assumed geologic conditions, and for these there is no cost consequence. If the probability of the occurrence of other end branch nodes is small (the geology is well-known and understood) or the cost consequences of other end branch nodes is otherwise very small compared to the cost consequence of the design set of parameter states (equal to the original design estimate or $K_{(n-2)} = T_{\ell}$ in Figure 4.13;) then the need to assess the uncertainty is slight. However, a well structured and logical design methodology is never remiss and therefore the use of the suggested approach will not be wasted.

Other methods of utilizing probability and decision theory may be even more suitable for application to urban mass transportation tunnels, especially if they include a model of the possible variation of the whole system rather than just geologic variations as presented here. The MIT rock model, Moavenzadeh et al (23,) mathematically simulates each subsystem and its interaction with a probabilistic geo-

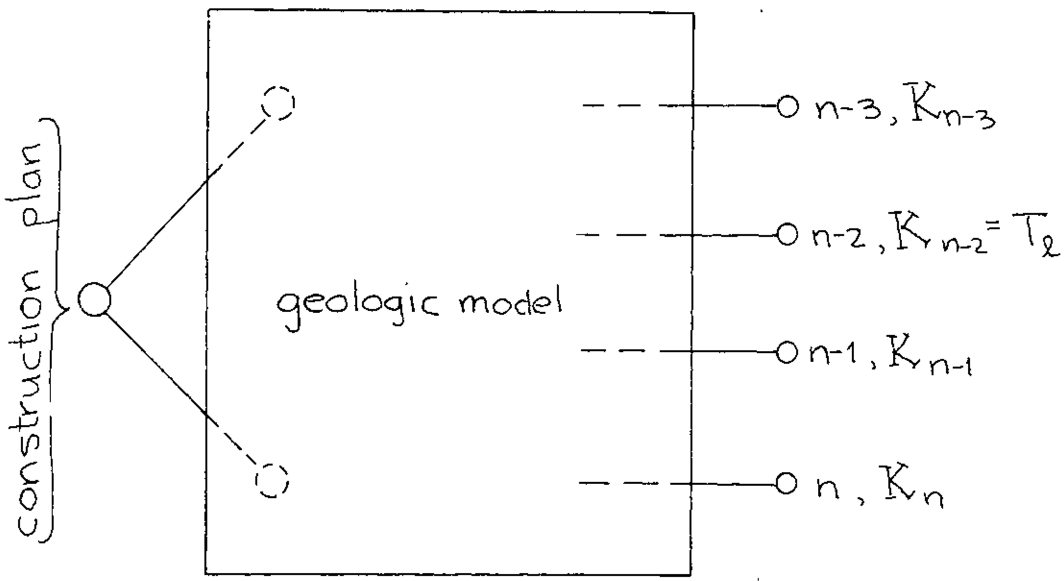


Figure 4.13 INITIAL SET OF GROUND CONDITIONS
WITH END BRANCH NODE n-2

logy and uses a computer to handle the great level of detail necessary to successfully simulate the tunneling process. Such a model can be of much more use to the contractor in planning his work than the simple cost model presented here because it allows his engineer to evaluate subsystem interaction in much more detail. Such detailed modeling also helps the engineer estimate the effect of geologic variation (and the effect of other uncertainties) on the system ROA which will lead to a more accurate prediction of cost consequences.

Meanwhile, any approach may be used as a tool as long as it is understood what price is paid for the additional advantages it may offer. The design system here attempts to look at the whole design/construction/environment interaction phenomena and does offer the designer many advantages.

1. It allows him to quantify his judgement concerning the effectiveness of a given design in a particular geologic environment, and allows him to estimate the occurrence of that one interaction mode.

2. It gives the designer a logical structured system within which to assess his design allowing him to minimize the effect of uncertainty on his judgement.

3. It gives him a means by which he can decide which are the key parameters in that particular design and gives him the foundation from which to make a decision on the appropriateness of further investigation.

4. He can incorporate his best judgement of the uncertainty of geologic conditions into his design giving him one more tool for making design decisions.

5. It forces the designer to look ahead to the construction to make him consider possible adverse occurrences and acquaint himself at an early stage with alternative construction strategies should these adverse events occur.

To conclude, there are two important points concerning this approach which must be kept in mind. First, the models presented here are not substitutes for engineering judgement or knowledge; but are tools for application by the engineer using all his experience and judgement to formulate the final design. Successful application of these tools is enhanced by an increase in the amount of pertinent data available concerning the geologic conditions and the skill of the engineer performing the design. Secondly, this approach, and the model particularly, are not magic boxes which translate data into final designs. The description of the approach in this thesis has not covered steps in formulating each design but on evaluating alternate designs. This allows the engineer the additional advantage of translating his opinion of geologic uncertainty into indices (EMV's) which can be used to evaluate alternatives and to choose the design which will be most adaptable and economical.

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