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# PROPOSED BASES FOR COLLATING EXPERIENCES FOR URBAN TUNNELING DESIGN

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*One could feel boundless surprise at the fact that Peck (ISSMFE, 1969) should state as recently as twelve years ago that the "first requirement for a satisfactory tunnel " is "that it should be able to be built". The statement is rather reminescent of the primeval steps of cognizance of any technology, wherein prevails the classical dualism "to be, or not to be ".*

*Multiplications and advances in the practices of design and construction of tunnels have been so proliferous over the past few years that at present we well recognize that in almost any situation it is no longer a question of being or not able to build a tunnel, but rather of assessing the feasibility thereof within a histogram of gradually varying degrees of difficulty, and wide spectrum of available techniques and solutions. We can build under almost any difficulties, but may we? It depends on the consequences, which have to be compared on the common donominator of costs.*

*Truly, however, despite much valid physical and geotechnical intuition on the art of tunneling and of assessing its problems, solutions, costs and consequences, we should recognize that at this moment the very Prescriptions that guided designers through their first steps are proving to be the greatest hindrance towards systematic objective analysis and synthesis for further progress. And progress is being generated, on the one hand by the creative engineering of men of construction and practice, while on the other hand men of theory undertake very respectable dialogues with Computers and finite element analyses without recourse to the least vocabulary of practice and realistic parameters.*

*It is the purpose herein to analyse some fallacies of present routine Prescriptions regarding soft ground urban tunneling, and to suggest that the origin of such oversimplified design prescriptions can be well understood: moreover, upon such recognition, the grounds for revision may be laid quite constructively.*

*While retaining our most grateful recognition of the great help given by leaders of our profession (Terzaghi, Szechy, Skempton, Peck etc.) and their Prescriptions, it is time for us to look constructively at the insufficiencies thereof, in order to profit of the exponentially vaster on-going tunneling construction, to collect and collate experience more fruitfully.*

## 1. Nature of the design/construction professional problem

It is necessary for us to begin by analyzing the nature of the professional problem on hand, as compared with other common soil and foundation engineering problems, and as thereby related to "Factors of Safety".

Soil engineers have intuitively recognized a dualistic distinction between: (a) problems wherein Design is well in control, and, associated with alert and dominant Inspection, should and can "assume responsibility"; (b) problems in which so-called Execution Effects predominate, and, no matter what the Design and Inspection acuities might predict and control, construction reality develops too unpredictably and fast. In the light of such distinctions one should honestly analyze the differentiated Contractual Obligations as forced upon Contractors through Bid Documents: and, as will be explained, possibly much of the hitherto unhappiness of owner-designer-contractor relationships may be understood, and even the probable forthcoming growth of such frustration may be foretold.

Of course, as an initial rejoinder one should recognize that in no case do Designers formally or really assume responsibility for any type of job: there is always in the lawyers' wordings of the General and Special Conditions the obligation for the Contrator to declare that he has acquired his own full knowledge (that nobody else has or can have any way) of the site, geology, quality of materials, etc.. etc..., and that thereby he a priori renounces any right to claim anything on the basis of insufficiency of data etc. Such leonine self-protective clauses are easily understood to have overflown from absurdities of the past. Firstly, Society is still imbued in the medieval distinction between "exact sciences" and "natural sciences", and civil engineering is conveniently categorized as an exact science; secondly, whereas medicine is favoured by the recognition of death as inevitable and the doctor's obligation as graciously postponing or alleviating the inevitable, and whereas lawyers are recognized to deal with 50-50 probabilities of guilt-innocence (and this probabilistic reality has been extended, by subconscious association, to a probabilistic acceptance of success-unsuccess in the cause defended), for the Civil Engineer failure is considered taboo, and directly punishable by "full responsibility" for the entire worth of the project. When the absurdity is subconsciously recognized, the "law" ends up never being applied. There are two gross incompatibilities that make present "laws" so inapplicable that regrettably the situation caters to proliferation of irresponsibility: one regards the gross financial incompatibility between a professional fee and the value of a project; the other involves a serious obstacle to the most intrinsic professional calling, of maximizing economy without an iota of a share in the result. While making a project as economical as possible (at presumed constant safety), the engineering fees generally do not change, whereas the Owner is the one who alone profits from the savings. "No taxation without representation" was an important historical call that may be recalled: Project benefits and risks go hand in hand and pertain to the Owner, while the engineers (designers and contractors) should be understood to risk their reputations and "fees".

It certainly cannot escape notice that the frustrations and claims in tunneling work are far more frequent than in common foundation work. If we do not assess the possible reasons for it, could it even occur that with improved techniques of sampling soils we might even inadvertently increase the cases of localized failures during construction?

In Fig.1 I attempt to demonstrate that whereas Soil Engineering has generally considered only one definition of Factor of Safety, FS, it can be important to recognize three distinct

Factors (considering only the differentiation of statistical dispersions around Resistances, R, without any further delving into the histograms of acting stresses S). Factor of Safety FS is a routine definition. I have chosen to call Factor of Guarantee FG the situation wherein by some lower rejection criterion I have assured myself that the histogram of Resistances can only be higher than some value already pretested or guaranteed. Obviously a value FG = 1.5 constitutes a much greater assurance of success than FS = 1.5. A pile jacked down under 60 tons to absolute stoppage of penetration/ settlement has FG = 2 if used for a working load of 30 tons: if the estimated resistance is 60 tons it has the conventional FS = 2. Setting aside the discussions on dynamic vs. static resistances of piles and cases of sensitive clays, driven piles checked by "refusal" observations can well be said to imply factors FG.

In contrast, a bored pile would suffer from two disadvantages in its load-settlement behavior. Firstly, it would never have been pretested, and therefore one might conclude that it is affected by the FS (poorer than FG). Secondly, upon closer examination we should reason that it is even worse than that. All efforts of advancement of Soil Mechanics are towards minimizing sampling and testing disturbances and better representing in situ soil parameters (intact soil elements). In reality the assessed intact parameters would establish an upper rejection criterion, since the soil affecting bored-pile behavior represents a histogram of resistances always lower, to varying degrees, truncated at the upper value. A situation diametrically opposite to that of FG, with the lower rejection criterion. One could denominate the new ratio of averages (Resistances/Stresses) a Factor of Insurance FI: insurance is against something essentially inevitable, that should be attenuated.

The basic fact is that $\boxed{FI < FS < FG}$ and depending on the dispersions of the histograms the differences may be very significant. If projects continue to be designed generally for (nominal) FS = 1.5 without recognition of this significant difference, all structures in which FI is at stake will record a much greater degree of troubles, while structures in which FG is at stake willincorporate an unnecessarily higher degree of safety.

Tunnels and bored piles involve execution effects that only deteriorate in situ parameters (resistance, deformation) and therefore involve FI conditions. We must examine to what extent the practical data supporting present Prescriptions would not force us to significant revisions merely because of recognition of FI vs. FS situations, and even further.

There is yet another important point to emphasize: the distinction between conditions which permit applying averages (as above), and those that involve localized situations corresponding to somewhere along the ends of histograms. That is, confidence limits and factors of safety might be related to "individual events on the histogram" (or fractiles) rather than on the median. Such is, for instance, the situation of instability of localized pockets along a bentonite-stabilized bored pile before concreting: after the concreting, the rigidity of the concrete guarantees applicability of the average over the profile. Similarly in a tunneling open face, during excavation localized instability may well be at play, corresponding to much more unfavourable conditions: behavior behind a steel face-plate of shield tunneling, or around a lining, can well be accepted to be averaged, which implies an inevitable benefit in comparison with localized worse conditions. A steel face-plate of shield tunneling, or around a lining, can well be accepted to be averaged, which implies an inevitable benefit in comparison with localized worse conditions.

2. Historical profiling of geotechnical parameters that are at Play

Very many have been the advances in soil profiling in the past decades. Typically there occurs firstly a cognizance of physical symptoms, related to adjectives and index parameters. Engineering progress may be typified by the statement: "we do, then we begin to explain and understand, and gradually we can and must quantify".

As regards tunneling design there were some truly remarkable simplifications of earlier times which should have been recognized but were clouded, and thereupon one could state that an intermediate step was temporarily thwarted: and, as often happens, the physical perceptions, categorized and simplified, were clouded by the very fact that for some time a pseudo-theoretical prescription diverted attention.

The problems were "cohesion" under lateral stress release, seepage, and "stand-up time". Strangely the emphasis of soil mechanics theorization, related to soft saturated clays under "quick" (undrained) loading (c. 1942-'60), dominated the picture so heavily that we could almost claim that for practical tunnel engineering (Peck 1969, almost to-date) it quite forgot the really dominant factors of stress release, seepage, stand-up time.

Fig. 2 presents schematically in the form of hypothetical subsoil profiles the parameters of cognizance recognized in the two arbitrarily quoted periods (c. 1946 and c. 1969) that represent reference milestones. In comparison, a present-day profile, shown side-by-side, would emphasize many obvious fundamental parameters of need. Foremost among developments of the past twenty years (post Boulder Shear Research Conference 1960 etc..) have been the emphasis on effective stress analyses and pore pressures (flownet plus due to shearing $\Delta V$), appropriate stress-path testing, recognition of the importance of pore-air (S%), recognition of the range of variation and importance of $K'_o$, and, finally, at the crest and in the wake of the computational wave, the "elasticity" parameters ($E$, $\mu$), and so on.

It has been contended repeatedly that once a theoretical reasoning establishes the backbone for a certain analysis-synthesis, the engineering method requires that we use that backbone for filling in the muscle and the trappings of experience. We cannot condone with Indices (either oversimplified, or complex-lumped-parameter) that do not fit into theorization, even if they may have been used as temporary struts. The fact that data (more specific or precise) are not available along the proposed line, does not excuse us from assuming the desired and necessary parameters: it only serves to expose the range of significance of our unknowns, and therefore, the technical and economic interest in seeking them. Meanwhile the engineer must, and can assume parameters as required, an can and must use approximations (often culled indirectly) for his working hypotheses.

In the three columns of Fig. 2, what stands out is our total neglect to-date of tests for design evaluation of "STAND-UP TIME".

## 3. Sequence of principal design problems

Merely for the purpose of elucidating the above rationale as an engineering technique, some crucial design questions of soft-ground shield tunneling in urban development may be listed.

3.1 - Face and excavation stability, and minimizing risks and consequences until the lining takes control.

Principal factors concerned with strength and stability may be listed as:

a) Face stability, and some known factors in controlling such stability.
b) Control of groundwater and principally its effects on face stability.

c) Penetrability of the head by jacking, and optimizing it in balance between increased safety vs. avoiding excessive transient stressing and subsequent increased consolidation settlements.

d) "STAND-UP TIME". Loss of strength from lateral stress relief and seepage stresses; loss of strength due to progressive shear straining (Sensitivity etc ...); loss of strength due to loss of capillary $u_c$; loss of strength of change from undrained (quick) to drained conditions.

Rates of such losses of strength; differentiation between rates of stress changes and rates of consequent strains.

3.2 - Thereupon, for the tighter conditioning of urban tunnels, the principal problems are those of deformations:

a) Predicting the settlement troughs that will be created in the ground at various elevations above the tunnel base, both "immediately" (before reaching the section and until a few days after passing it), and with time (consolidation, creep, etc..);

b) Deducing the consequent tendency towards differential deformations (settlements and displacements) of nearby shallowly founded buildings (assumed "flexible"); in the case of nearby deep foundations, deducing the incremental loadings (negative friction and transverse loading), and consequent deformations transfered to bases of columns;

c) Establishing quantifiable indices for acceptable or tolerable damage threshholds for such buildings;

d) Optimizing soil-structure interaction of the tunnel lining itself so as to minimize rigidity (moment reinforcing) without aggravating the differential deformations of 3.2 (b).

4. Face stability

It is doubtless one of the most serious problems. In advancing a tunnel excavation we face a temporary condition of different degrees of proximity to provoking a failure at face and/or roof. Moreover it is particularly critical because of always advancing into the unknown and facing non-averageable localized conditions (SI of individual case or fractile). To avert or attenuate this unpredictability, various procedures of advance pobing are in use, recommended, and valid: and in Fig. 9 we hint at the complementary advantage that may be gained if the tip of the boring could be installed as a drain.

The "stability" involved has been associated almost exclusively with a "cohesion" value (historically and still generally deduced from unconfined compression tests, in the case of plastic saturated clays in which it is presumed that the UU or Q strength envelope is $s = c \simeq 0.5\ q_u$). Routinely one is led (Peck, 1969) to look for a Stability Number (Broms and Bennermark, 1967).

$$\frac{\gamma z - p_a}{s_u} \not> 5 \text{ or } 6$$

$\gamma z$ = total vertical pressure at depth z of center of tunnel
$p_a$ = air pressure above atmospheric
$s_u$ = undrained shear strength of clay

The Broms and Bennermark (1967) paper, which follows closely the Bjerrum and Eide (1956) paper, clearly represents a significant contribution for its time and for the very specific idealized problem evisaged. It concerned saturated plastic clays ($s = c$, $\phi = 0$ undrained), normally consolidated (overburden total $\sigma_V$ as the principal driving stress), and clearly demonstrated the association of the face stability with a bearing capacity formulation, $cN_c$. In subsequent discussions herein we shall limit ourselves to simple bidimensional conditions in order to elucidate comparative conditions at play. In the same way as is generally done in bearing capacity formulations, the circular face stability can be estimated from bidimensional formulations by use of adjustment factors and shape factors (often extracted from analogous situations).

The Broms and Bennermark tests were literally extrusion tests. There is the (conservative) assumption that failure caused by increasing $\sigma_V$ would preserve the same maximum deviator stress (function of $q_u$ and cohesion), as failure caused by decrease of $\sigma_H$: the decrease of internal $\sigma_H$ was simulated by an increase of $\sigma_V$ external. This assumption is idealized, because in practice there is a tendency to compress and generate positive pore pressures in the first case, whereas in the second, any tendency to expansion at the face would immediately create capillary tensions. There is a significant question regarding the method used to simulate confining pressure: "Confining pressure was used to investigate the effect of compressed air to prevent a cohesive material from flowing into an excavation or tunnel. Glycerin was used as a confining fluid".

The important influences of capillary tension and of differentiated interstitial pore fluids and liquid-liquid surface tensions had merited some attention in the early 1950's. Unfortunately, however, they are generally eliminated in idealized laboratory conditions, and/or often overlooked. Some representative data are summarized in Fig. 3, just as a reminder. The special importance of compressed air at a tunnel face cannot be dissociated from some capillary minisci, and the fact that soils generally are not fully saturated. Depending on the magnitude of the air pressure, in fact there can be a favourable reversal of flow direction, and consequent favourable seepage pressures to complement the favourably propagating capillary tensions.

In the submerged saturated clayey sands of São Paulo laboratory tests indicated that although under very small gradients (about 0.2) practically no change of moisture content W% was caused (about 0.2%), under much higher gradients (up to 30) decreases $\Delta W$ up to 6% were achieved in less than 1 hour. The graphs of variation of unconfined compression strengths with W% are given in Fig. 4 a,b. As is well recognized, complete drying is unfavourable. But the benefits of somewhat higher air pressure (and local gradients at critical points) are so evident, that it need hardly be emphasized that there is direct and simple and beneficial cure for face drying of a sand: one need but spray the face with moisture, preferably muddy (dirty) water.

The first basic fact regarding failure under stress release is that, as a general principle, materials exhibit loading-unloading hysteresis (in greatly varying degrees), and, therefore under conditions of unloading there is always some "cohesion intercept" and $\phi = ds/d\sigma$, however small and/or temporary. When we deal with so transient a condition (tunnel face excavation) so close to FS = 1,0,one cannot afford to neglect these minute components in comparing successful vs. unsuccessful experiences.

One adjustment factor that could be applied to the $s = c$, $\phi = 0$ Stability Number, in consideration of an applicable $\phi$ value has been suggested by Rebull 1972. The comparative influence is indicated in the graph of Fig. 4(c). Other such analyses may be available and/or forthcoming. However, unless an analysis can really begin to take into account problems of pore pressures, seepage, $K'_o$ as dominant parameters, it is not likely to facilitate appropriate comparisons.

Merely as an example of methods of working analyses available for assessing comparative solutions, we present a series of cases analysed on the basis of flownets and effective stress envelope. Firstly, it is emphasized that the flownets and analyses have been prepared for the two-dimensional condition (as a liberty, merely to exemplify). Fig. 5 shows the estimation of the adjustment factor that could be deduced in a simplified manner for the transfer of bidimensional to three-dimensional data as regards flownet pore pressures.

The next figure (Fig. 6) indicates schematically for hypothetical failure surfaces how the in situ undrained strength has been estimated, taking into account only flownet u values and $K'_o$, applied to overburden $\sigma'_V$ and an effective stress envelope. It is recognized that in principle there can be a need for correcting the flownet u values because of tendencies to $\Delta u$ as a function of shearing $\Delta V$: judgment may be applied for such corrections, in the light of a feel for the material's behavior and the probable stress path. No matter what failure surfaces may be analyzed, it cannot escape notice that the Stability Number can vary most widely depending on u and $K'_o$.

In the following figures (Figs. 7, 8, 9) we have sketched rough two-dimensional flownets for some of the conditions typically encountered in tunneling, and in methods used to control seepage pressures. The purpose is merely comparative. In the hypothesis of a slightly excessive compressed air pressure, for a short transient condition, it is assumed that there is essentially a reversal of the water flow in the saturated soil within a laterally confined variable section, therefore with the same pattern of flowlines and equipotentials.

Finally in Fig. 10 we summarize the comparative "mass statics" that should give a feel of the influences of different drainage and/or compressed-air treatments. Assuming that the resultant $\sum(u) = U$ values on the failure surfaces (rigid body statics based on total stresses and boundary neutral forces of membrane hypothesis) are the key to the overall stability problem, the comparison is based merely on these values.

For the present comparisons (rigid body with boundary neutral pressures) the artifice is used of reduction of the horizontal force to zero by "transfer of axis", because the real beneficial effect of the compressed air is to reduce (or occasionably even invert) the effective stresses due to seepage. The results indicate trends only, because we must carefully distinguish between artifices employed for analysis of the statics of "rigid bodies", and the extent to which the "effective stress behavior" only sets in to the point that corresponding strains (compressions and expansions, void ratios) have materialized. In a perfectly saturated ideal clay the undrained instantaneous changes of pore pressures do not generate any changes of in situ strengths.

Many an important conclusion, intuitive in tunneling practice, may be drawn, not only regarding the overall "rigid body statics" but also regarding locally critical failure conditions. These are affected principally either by stress release of the higher horizontal stresses (with overall tendency to expansion and loss of strength concomitant with the principal stress reversal) but also due to positions of more critical seepage exit gradients, and corresponding tendencies to expansion, loss of strength, and failure. Such localized conditions may be approximately analyzed by Mohr circles. Depending on such localized conditions, the undrained stability solutions based on the bound theorems of plasticity may fail to reflect any semblance of realities faced in the field.

As a concluding comment concerning face stability it must be emphasized that the problem matters no only as regards the transitory stability itself, but also as regards settlements. As is well known, deformabilities increase significantly as the FS decreases. Soil Engineering is not documented with plate load tests (compressive) on faces of test pits, although it is a test with much use for transverse loads on piles etc.., and is a test pregnant with practical possibilities. A fortiori, one finds absolutely no data on unload-deformation of plates supporting vertical faces (analogous to convergence observations across diameters of tunnels). If and when such data become available, they could be plotted in a manner similar to that adopted in Fig. 11, wherein we have analyzed the foundation plate load tests of several São Paulo soils. The very rapid decrease of E as one approaches "failure" is as expected. One suspects that under stress-controlled "soft-load" conditions the unloading behavior will possibly show an even sharper drop of E in the lower FS range.

## 5. Penetrability of the head by jacking

The incremental stability given by an appreciable advance of the head is obvious from a structural point of view (cf. forepoling), and also because of the improved seepage control. On the other hand, jacking to high loads has its obvious disadvantages, because of stressing the soil ahead (with consequent pore pressures and additional settlements), and remolding or fissuring the soil being penetrated. It should therefore be optimized. One finds almost no data on the problem. Once again, the use of a static penetrometer in test pits, for horizontal penetration by jacking against the opposite face, would seem a simple and very practical test. The fact is that in a delicate field of immense responsibilites, that of ocean platforms, the skirt penetrations are being reasonably predicted from CPT results (Zide et al. 1979, Kjekstad and Lunne, 1979).

## 6. Prediction of settlement troughs

Comprehensibly the estimation of the settlement trough constituted the second principal hurdle at the time when Peck (1969) offered his great contribution towards mentally organizing the advances of the then strictly empirical art of tunneling, for the purposes of making them amenable to a minimal geotechnical engineering treatment. So it was, therefore, that as has happened so often before, the profession owes much gratitude to the fact that a man of stature was willing to step into the vacuum to offer: (a) as a first stilt, a PRESCRIPTION, that of a Gaussian settlement curve (earlier postulated by Litviniszyn, 1955), with the admonition "although the use of this curve has no theoretical justification, it provides at least a temporary expedient" (p.240); (b) the qualitative indications of the principal intervening factors; (c) the summary table of "all" the data available, with the candid confession that "the information ...... is surprisingly meager", and with the appropriate call for "full-scale field observations".

It is herein contended, however, that the collection of data calls for a mental model, and we must urgently set aside in totum the unfortunate association with a Gaussian curve, because it is a dead-end road and carries no idea fertility. We must foster some minimum theoretical analysing on the different parameters asciatable with the full-scale field observations, since progress in design procedures and predictions will only be achieved if we set about to dispell the unnecessarily pessimistic forecast "Because of the dependence of loss of ground on construction details, there seems little likelihood that theoretical investigations will prove fruitful except for some of the simplest of materials such as plastic clays" (p.245). Although PRESCRIPTION do constitute the valid base for design developments and decisions, they must be rapidly adjusted by statistical CORRELATIONS on observed behavior in order to permit revision and progress. And we must make an effort to resist the widely spreading practice of statistical regressions at random, since a statistical correlation is meaningless and can be dangerous unless it is based on theorization on the physical model, to establish the nature of the equation and its coefficients.

Surely it is accepted that in tunneling we automatically face a greater proportion of strictly localized conditions of heterogeneity and possible failure (loss of ground), as has been emphasized regarding SI of individual points or fractiles on a histogram. Such conditions are those that must either be bearable and borne as risks unquantifiable, or must be resolved in design and construction by "a change of statistical universe" (i.e. a treatment that essentially excludes the problem). Our design engineering concern can only be with conditions that permit averaging, and quantifications based thereon. The fact is that settlements most often distribute well enough to validate statistics of averages.

Fig. 12 summarizes the Peck 1969 prescriptions regarding the settlement trough. The basic points are: (a) geometry, dimensions; (b) a Gaussian curve of settlements and no indication of displacements; (c) a graph plotting the available observed data (a point for each casehistory) with reference to index classifications, irrespective of association with geotechnical parameters.

The presumed Gaussian curve is really that of pseudo-elastic and/or elasto-plastic changes within the semi-infinite mass.

Such is the nature of the phenomenon at play when tunneling design and construction proceed under normal conditions, with minimized, erratic defective occurrences. There is absolutely nothing probabilistic or stochastic about it. Indeed, for local critical occurrences (cave-ins etc.) there are probabilities of occurrence along the tunnel: but one hardly could predict, or presume, or even establish a posteriori the frequency distributions of such occurrences for the longitudinal advance of the tunnel (which, moreover, would most often represent a perceptibly varying geomechanical universe, and not random variations within a presumed constant universe).

It is, indeed, strange that a probability phenomenon and function should ever had suggested itself. Litviniszyn analyzed the subsidence that would be caused in a loess if there were a local underground collapse or cavity: representing the material (considered a discontinuous, rigid bodies, separated by cracks) as a mass of uniform spheres, and visualizing the cave-in as the downward movement of one sphere, he obviously concluded that the subsidence profile at surface could be represented as a Gaussian probability. The result is mathematically inevitable. Two phenomena that under idealized conditions lead to the same equation are not thereby similar phenomena.

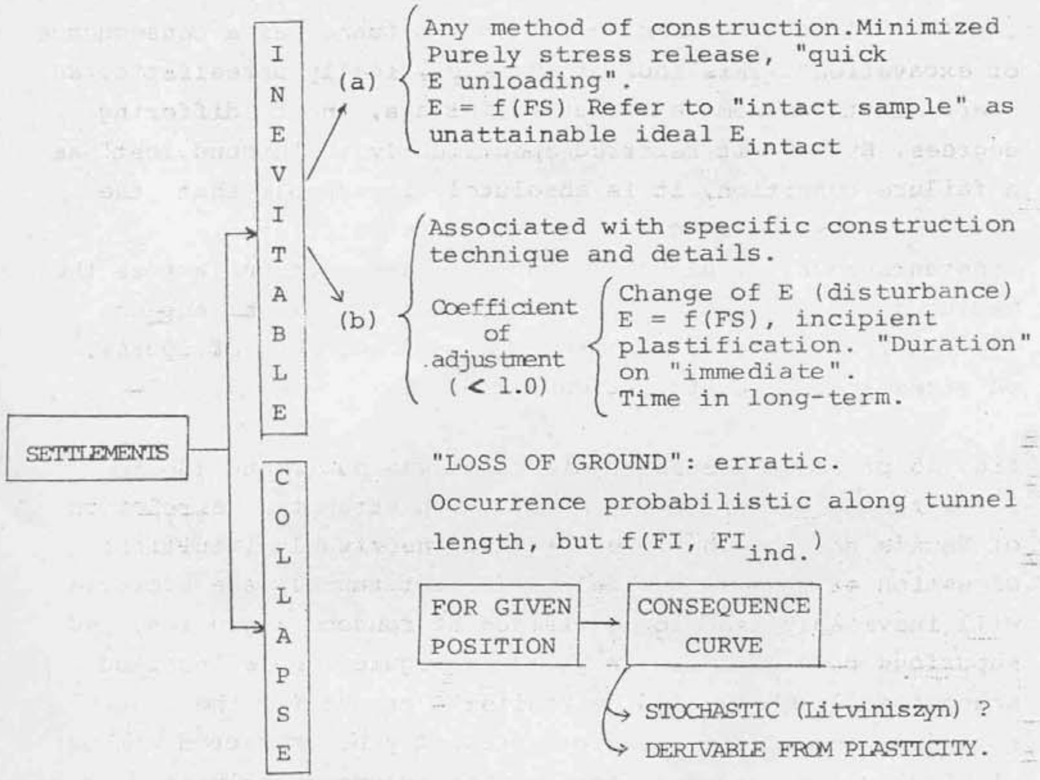
There is many a situation where, after making the necessary simplifying assumptions (usually averaging, and Gaussian) the mathematical equations of a given physical phenomenon become identical to those of many other totally distinct phenomena: for instance, the classical similarities between Darcy-Laplace seepage flownets and the electrical analogy models, or arrangements of iron-filings within appropriate magnetic fields. It would be absurd, however, to follow up with a dogmatization on the mathematical result (idealized) to insist on fitting experimental or observational data of the first phenomenon into the equation of the second: for instance, when capillarity intervenes in the flownet result, it surely is not against the electrical analogy models that one should force the data-fitting.

Peck well emphasizes that "every soft-ground tunnel is associated with a change in the state of stress in the ground and with corresponding strains and displacements", and therefore it is surprising that Litviniszyn's formulation should have detracted from a direct association with stress-strain changes in a pseudo-elastic medium (cf. Fig. 13), especially in view of Carrillo's early brilliant contribution "Subsidence in the Long Beach-San Pedro, Cal. Area: the effect of a tension center" (1949). The principal problem, in my view, has been the early confusing use of the term "loss of ground", and the tunneling foreman's intuitive feel that settlements (i.e. big, most noticeable settlements) derive from loss of ground. Since in practice one's attention first concentrates on immediate cause-effect evidences, and especially on failure, the primeval confusion is understandable. However, it has nothing to do either with engineering quantifications, or with the "representative points" (without even a width of dispersion) plotted from data tabulated by Peck (and by most authors).

In fact, even for the "collapse of cavity" condition it should be recognized as much more conducive to fruitful experience collection and collating, if instead of adopting a geomechanically sterile stochastic postulation (dissociated from parameters physically comprehensible and derivable) authority had fostered resorting to plasticity formulations ("collapse of cavity" as an inverse of the widely recognized solutions of "expansion of a cavity in an infinite medium").

The most curious fact is that the fostering of the Gaussian curve design prescription predominates among the self-same Design Companies that are most eager to spread the use of Finite Element Analyses for the same problem whenever the shape of the cavity differs from the circular, or whenever in Rock Mechanics there is opportunity to insist on the problems of internal stresses. A single example (cf. Fig. 14) is sufficient to illustrate the obvious.

Peck's candid recognition (p. 231) "It is not yet possible .... to apportion the lost ground between the inevitable movements associated with a particular method of construction, and the additional movements that may arise because of poor workmanship or faulty techniques" make it imperative to examine (statistically) the <u>varying</u> $K'_o$, FS, E (etc.) <u>conditions along each tunnel</u> (constant construction technique universe) in order to separate, as in hydrology, the "peak flows from the base flow".




In this respect it could be a sadly moot question whether it is an advantage or disadvantage that the Litviniszyn collapse formulation should lead to exactly similar distribution of settlements as the elastic and elasto-plastic solutions. Widely different distributions could be sorted out. But, how could they be different, if the stochastic formulation represents nothing but a mathematical abstraction for conditions so idealized as to give the anticipated physical behavior? In short, the Gaussian prescription must be escluded in limine because it is sterile.

There is one additional point of greatest relevance to design. Peck (1969) would give us the shape of the curve, but no direct help in establishing the predicted maximum settlement of each section, directly above the crown. There was a first-order indication "Measurements have established within reasonable accuracy the equivalence of the volume of surface settlement and the volume of ground lost into the tunnel as a consequence of excavation". This indication is physically unrealistic, as there has to be some attenuation, always, and to differing degrees. Even if it referred specifically to "ground lost" as a failure condition, it is absolutely impossible that the Volumes transmitted across the medium should, even "instantaneously", be equivalent. The attenuations across the medium have to depend very much on the FS at face, and on the $\Delta E/\Delta FS$ at face and across the medium, and, of course, on stress-strain distributions.

Fig. 15 presents the indication that was published (Souto Silveira and Gaioto, 1969) based on an attempted correlation of Peck's data, without recourse to theorizable intuitions. Digestion of data from widely different tunnel case histories will inevitably lead to statistics at random, confusion, and supurious correlations. In the same Figure I have inserted schematically what could be realistic trends for the correlations: these curves can presently be extracted without difficulty from elasto-plastic finite element analyses.

Finally in the same Figure I have schematically indicated that even assuming unchanged geotechnical behavior parameters, there is a net difference between considering the face-plate support (or membrane boundary loading) and the realistic use of body stresses, effective stresses due to gravity composed with those due to seepage. Deformations are not equivalent. The routine computational artifice is perfect for rigid body statics. As minute deformations and differential deformations have become important to buildings, this source of divergences of behaviors and opinions must be considered.

7. Minimal considerations regarding effects of ground deformations on buildigns

There has been a tendency to apply automatically the Skempton - McDonald (1956) and Bjerrum (1963) prescriptions on limiting allowable differential settlements, to judge on consequences of urban tunneling settlements to the fissuring or cracking of wall panels of buildings. Once again, we begin by gratefully recognizing the immense debt of the profession for those first PRESCRIPTIONS. But it is tragic to observe how much subway construction all over the world has foregone the opportunity for really developing valid statistical correlations.

We shall not expatiate on the many differences between the settlements and differential settlements observed as a building rises, and the ground settlements an existing building is forced to face. As was pointed out in 1969 (de Mello), in settlement computations of buildings, design decisions are based on computed soft load (flexible building) differential settlements, but their effects (tolerated or not) that reflect in incipient or evident cracking ipso facto signify redistributed "hard loads" and settlements: therefore, the limits indicated from cracked observed buildings are not directly equivalent to anticipated design differentials. Moreover, the early analysis of data for the prescription suffered from two problems: (a) widely different universes, each building being a case in itself (b) the attempt to condition to a difficult criterion, of "initial cracking" (analogous to extreme value statistical conditions).

In urban tunneling the ground deformations are (for the shallow foundation building) real, and sudden: moreover, they are incremental differential deformation on an unknown initial condition. In short, they suddenly remind us that initial conditions are never really known(not even for "new buildings"). It is technologically sterile to concentrate attention on conditions close to initial, close to the start. What we can determine very suitable, is the incremental cracking vs. incremental differential settlement,after the first fissuring has begun. It is extremely practical, economical, and the only realistic approach for collecting and interpreting data. And with the advantage that each building can be treated (for closer analysis) as the reasonably constant statistical universe it really is.

After establishing graphs with great númbers of points of $\Delta$ crack/ $\Delta$ (diff.settl.) we could extrapolate backwards or forwards at will to the desirable prescriptions of acceptable limits.

For the observation of when the first fissuring begins we can rely on a vast number of observers, gratis: all the occupants of the building, and visual observation. Once the fissure has started, the observations of the settlements of the contiguous columns can begin, alongside with observations of the widening and lengthening of the crack (and additional cracks).

In short, here again, for the purpose of developing more useful design prescriptions it is necessary to conceive a mental model for the very collection of data.

8. In closing it should be emphasized, with our deepest respect and gratitude for the fruitful contributions that helped us to this point, that it lies in the glorious destiny of a fruit that it should mature, fall, and rot, so that from its seed may grow another tree for further fruitage.

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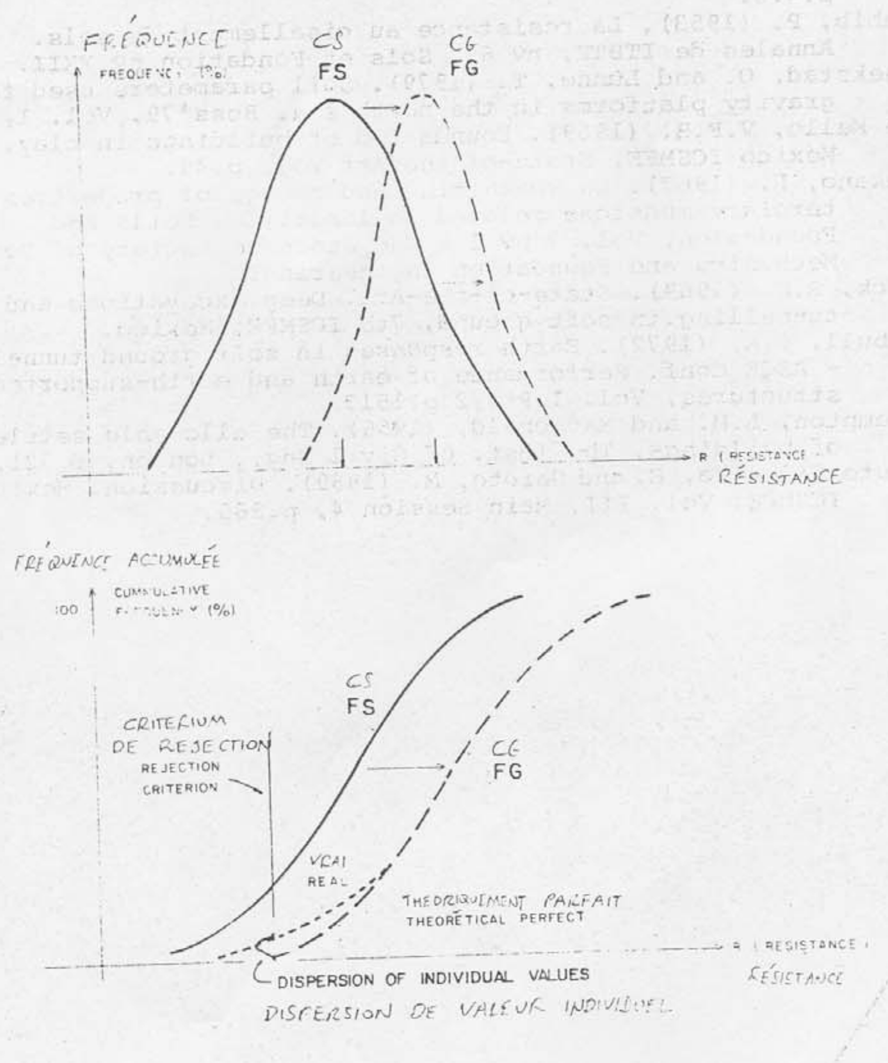



PROPOSITION POUR DISTINCTION ENTRE COEFFICIENT DE SÉCURITÉ (CS)

FIG I A PROPOSED DISTINCTION BETWEEN FACTOR OF SAFETY (FS) AND FACTOR OF GUARANTEE (FG)

E COEFFICIENT DE GARANTIE (CG)

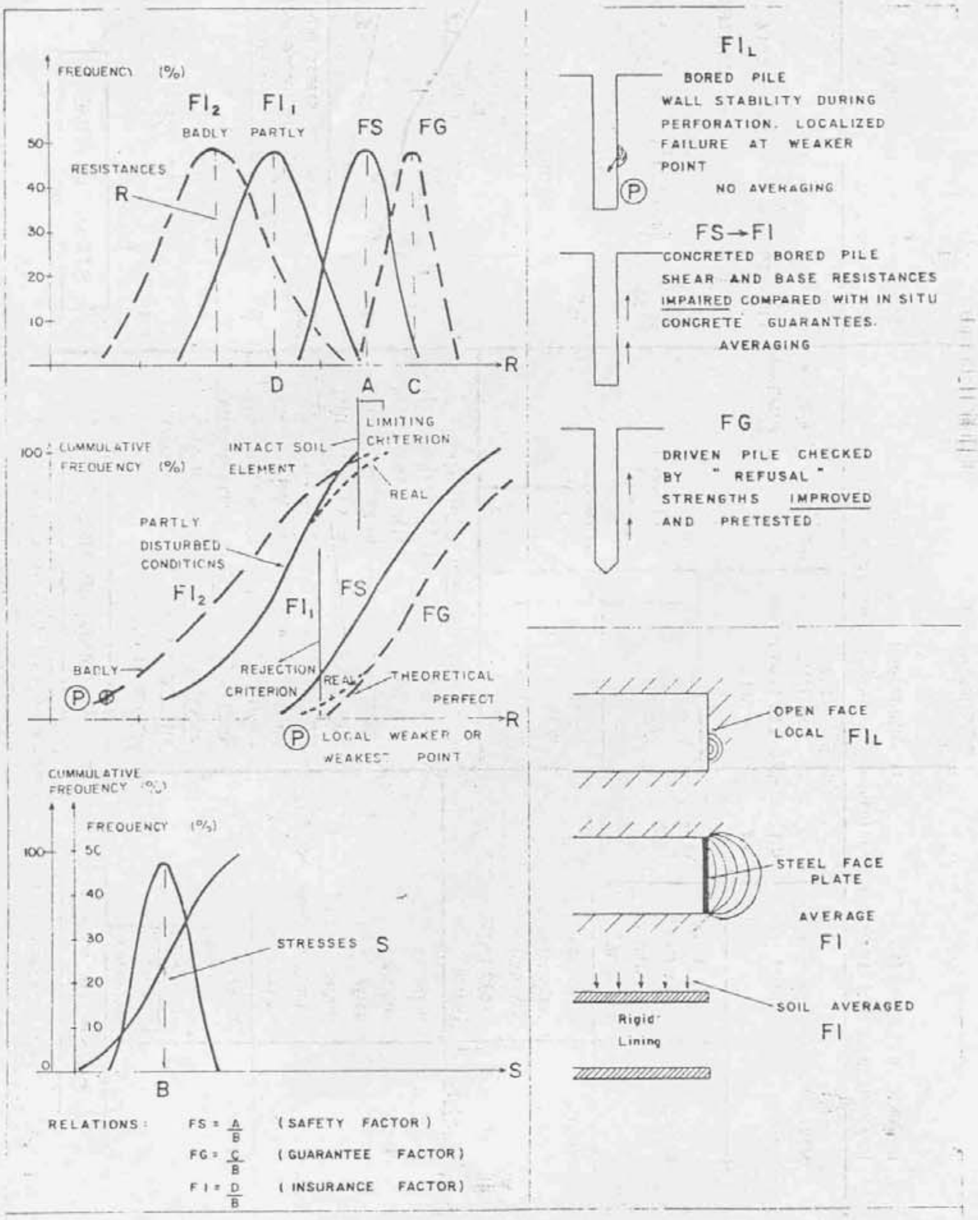



FIG IB SUGGESTED DISTINCTIONS IN "FACTORS OF SAFETY"

**HISTORIC FROM TUNNELING PRACTICE (apud Peck 1969; Terzaghi 1946)**

| | | |
|---|---|---|
| SQUEEZING SOILS | VERY SOFT TO MEDIUM CLAYS | W.L. NOT OF CONCERN (?) |
| FIRM GROUND | STIFF CLAYS, CEMENTED OR COHESIVE GRANULAR. | |
| RUNNING GROUND | PERFECTLY COHESIONLESS; DRY SAND; CLEAN LOOSE GRAVEL | |
| RAVELING GROUND | SLIGHTLY COHESIVE SANDS, FINE SANDS, SILTS (APPARENT COHESION); SAPROLITES. | |
| FLOWING GROUND | RAVELING OR RUNNING GROUND WITH SEEPAGE PRESSURES | |

STAND - UP TIME ?

**TRADITIONAL SOIL MECHANICS, STATE-OF-THE-ART (PECK 1969)**

| | |
|---|---|
| SENSITIVE CLAY (INCL. LEDA) NORMALLY LOADED | (cf. TABLE 1. Ref.) $C = \frac{1}{2}$ qu (LOADING $\Delta\sigma_1$) (MODERATELY UNDISTURBED SPECIMENS) $\phi_{UU} = 0^0$ |
| PLASTIC GLACIAL CLAY | |
| FISSURED PLASTIC CLAY (LONDON, BOOD) | SATURATED W.L., U, $K'_o$ ? |

STABILITY IN MOST SOILS ....

(P.228) "WITH THE EXCEPTION OF PLASTIC CLAY SOILS UNDER UNDRAINED CONDITIONS, THEORETICAL ATTEMPTS TO ESTIMATE THE FACTOR OF SAFETY ... OF A HEADING HAVE NOT YET BEEN SUCCESSFUL" ..... "DETAILS OF STRATIGRAPHY AND .... SECONDARY STRUCTURE OF THE SOIL DEPOSIT. SINCE THESE DETAILS ARE UNPREDICTABLE, NO SATISFACTORY CORRELATION BETWEEN F.S. AND MEASURED SOIL PROPERTIES CAN BE ANTICIPATED".

STAND - UP TIME ?

**PRESENT MULTIPLE PROFILING ON GEOTECHNICAL DATA**

SPT — estimated s

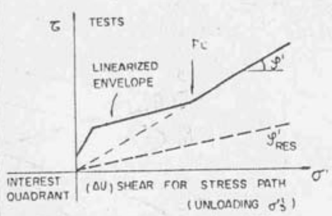
CPT, $CPT_U$ — estimated s, E, $C_h$

UNDISTURBED — PERFECT — INTACT ELEMENT

CLAYS

S$ — $p_c$, $C_c$, $C_v$; OCR; $K'_o$ (±)

W.L. + U SEEPAGE




SANDS, CLAYEY SANDS, SILTS, ETC.

S$

W.L. + U SEEPAGE

ESTIMATED — $D_r$: E: $\phi'$, c'; $p_c$: $K'_o$

STAND - UP TIME ?

FIG. 2 - IDEALIZED SUBSOIL PROFILE AND PARAMETERS ESTIMATED FOR FACE STABILITY

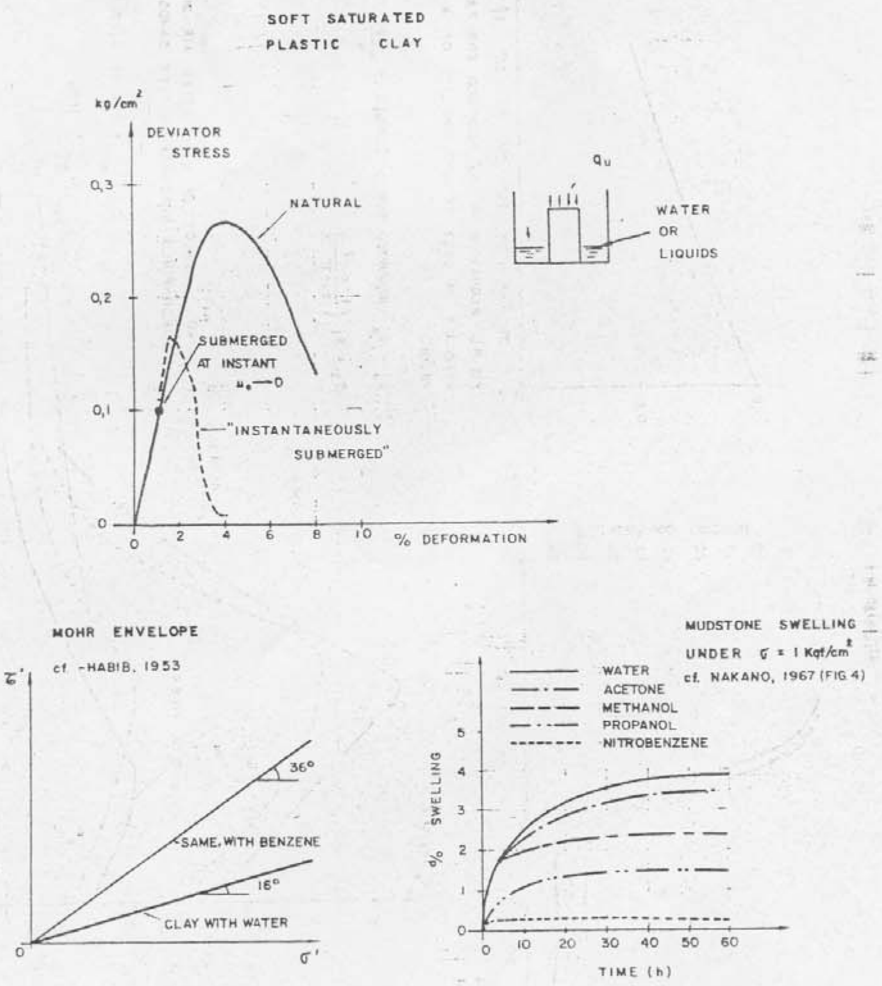



FIG. 3 INFLUENCE OF $u_c$ AND DIFFERENT LIQUIDS IN UNCONFINED COMPRESSION

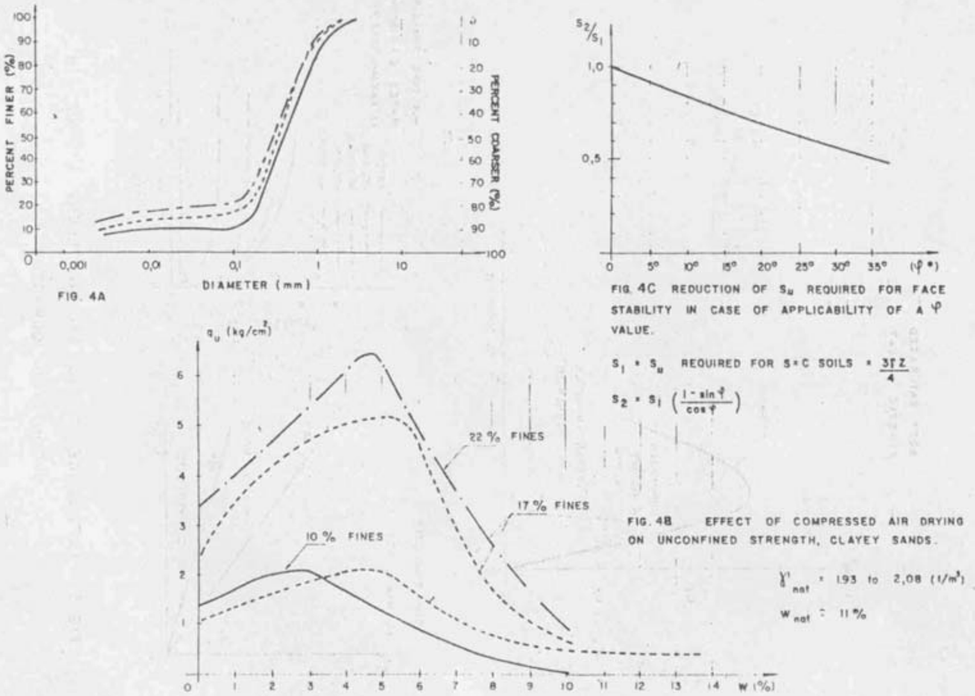



FIG. 4A

FIG. 4C REDUCTION OF $S_u$ REQUIRED FOR FACE STABILITY IN CASE OF APPLICABILITY OF A φ VALUE.

$S_1 = S_u$ REQUIRED FOR $S = C$ SOILS $= \frac{3\gamma Z}{4}$

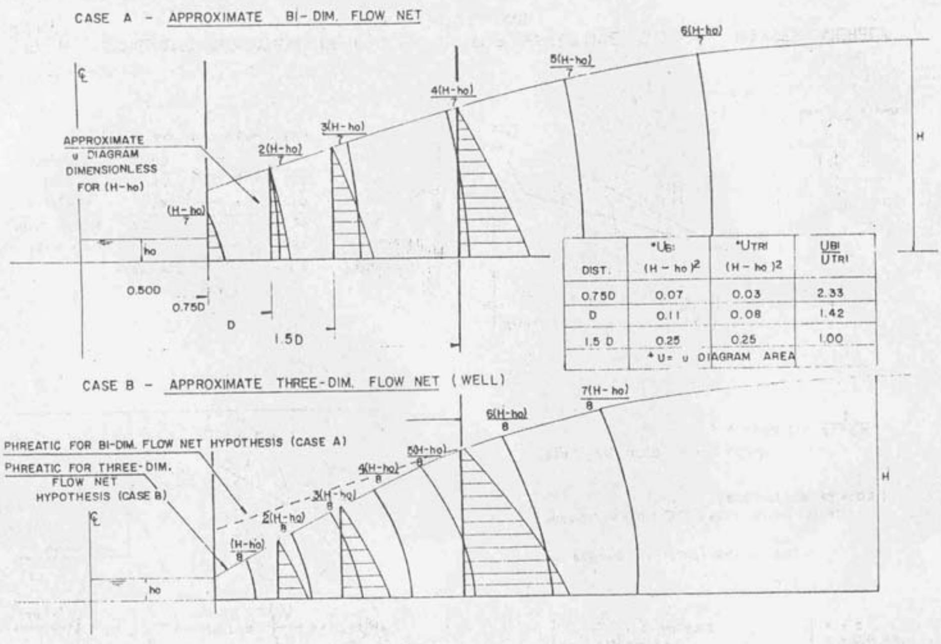
$S_2 = S_1 \left(\frac{1 - \sin\varphi}{\cos\varphi}\right)$

FIG. 4B EFFECT OF COMPRESSED AIR DRYING ON UNCONFINED STRENGTH, CLAYEY SANDS.

$\gamma'_{nat} = 1{,}93$ to $2{,}08$ (t/m³)

$w_{nat} \simeq 11\%$

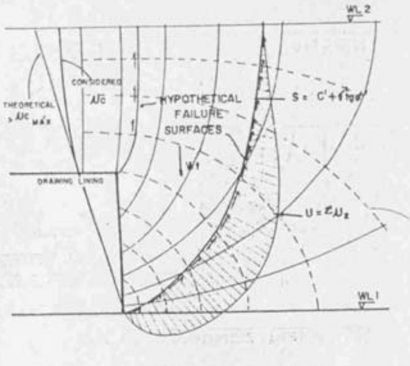
FIG. 4 DATA ON UNCONFINED COMPRESSION OF CLAYEY SANDS, SAO PAULO. (FIG. 4A, 4B) INFLUENCE OF φ ON FACE STABILTY (4C)




| DIST. | $^*U_{BI}/(H-ho)^2$ | $^*U_{TRI}/(H-ho)^2$ | $U_{BI}/U_{TRI}$ |
|---|---|---|---|
| 0.75D | 0.07 | 0.03 | 2.33 |
| D | 0.11 | 0.08 | 1.42 |
| 1.5 D | 0.25 | 0.25 | 1.00 |

* U= u DIAGRAM AREA

FIG. 5 ASSUMING TRANSIENT UNAFFECTED PHREATIC AT 1.5D AHEAD OF FACE




1st CASE: WL1
$U = U_c$
$K_0' = 0.5$ and $K_0' = 2.0$
$s = C' + \sigma' tg\phi'$ and $s = C$
C VARIABLE

2nd CASE: WL2
$U = U_{flow\ net}$
$K_0' = 0.5$ and $K_0' = 2.0$
$s = C' + \sigma' tg\phi'$
$s = C$

Wt REMAINS CONSTANT: ONLY CHANGES U

NOMINAL $s = C = \frac{Rc}{2}$ ( PECK, BROMS AT AL ASSUMED FOR $K_0' = 0.5$ )

REAL, PROPOSED $s = C' + \sigma' tg\phi'$ ( USING $\sigma' = \frac{\sigma'_V + \sigma'_H}{2}$ )

| | $K_0'$ ASSUMED | U | F S | REFERENCE |
|---|---|---|---|---|
| PECK etc | 0.5 | —— | 0.57 | 100% |
| TOTAL STRESS | 2.0 | —— | 1.20 | 210% |
| EFFECTIVE | 0.5 | from flow | 1.10 | 190% |
| STRESS | 2.0 | net U=42 t/m | 1.77 | 310% |
| ANALYSES | 0.5 | Uc | 1.60 | 280% |
| | 2.0 | U=220 t/m | 2.88 | 500% |

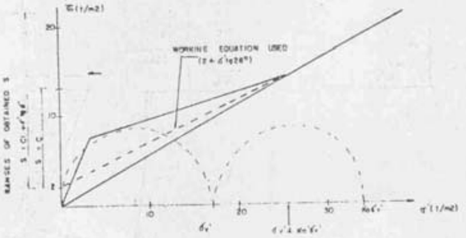



FIG. 6 SCHEMATIC INDICATION OF PROCEDURE FOR EFFECTIVE STRESS ANALYSES MERELY FOR COMPARISONS

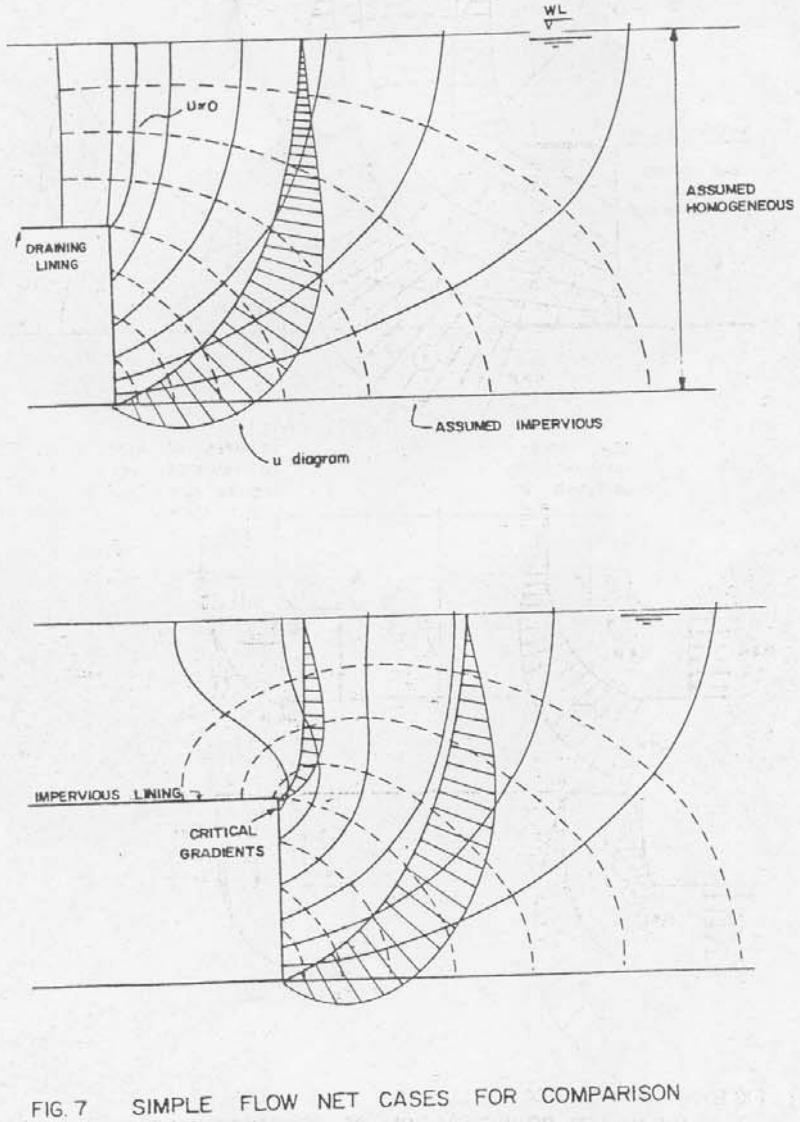



FIG. 7 SIMPLE FLOW NET CASES FOR COMPARISON

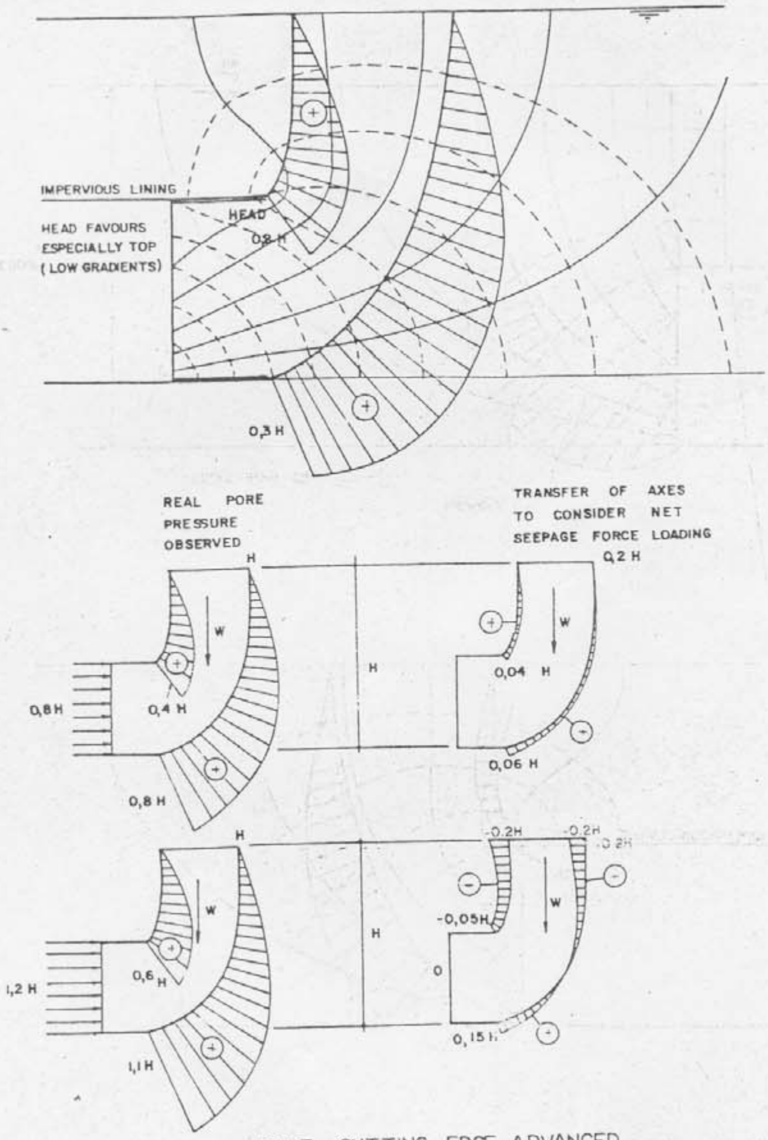



FIG.8 ADDITIONAL CASE, CUTTING EDGE ADVANCED. IDEALIZED CONSIDERATION OF COMPRESSED AIR.

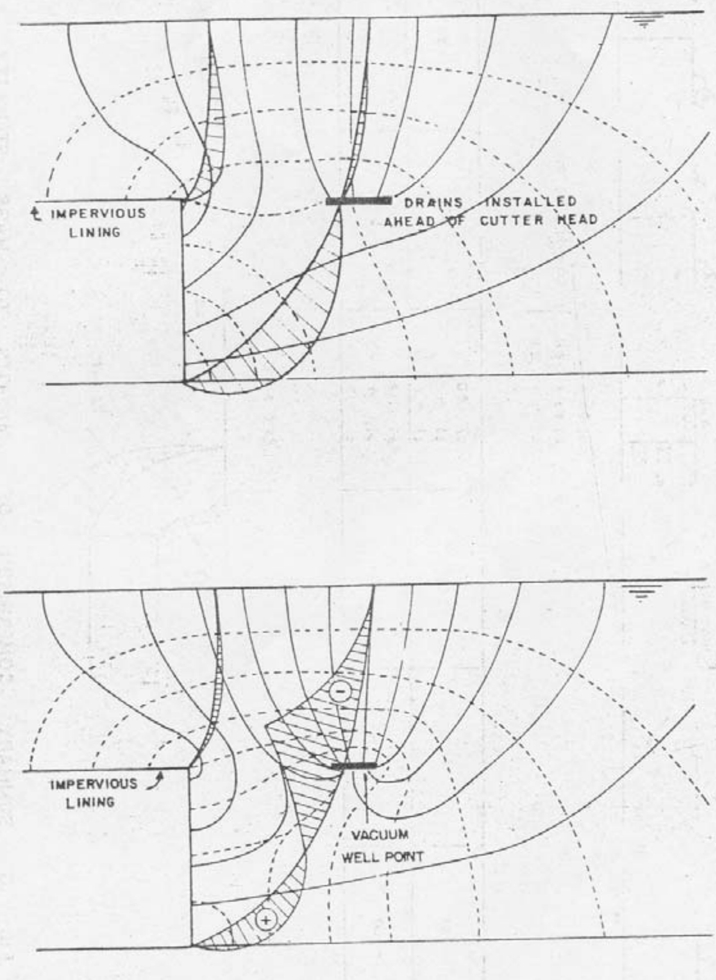
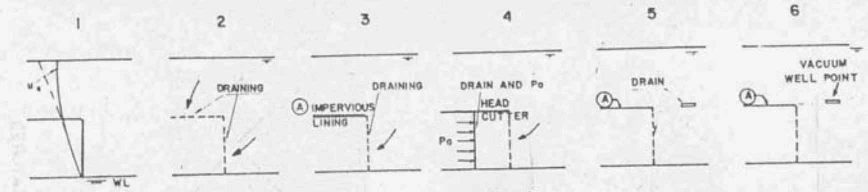
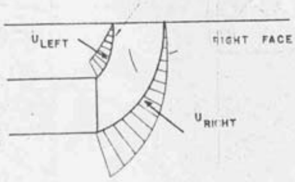



FIG. 9 ADDITIONAL CASES SPECIAL DRAINAGE FEATURES AHEAD OF FACE




| | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| $U_{RIGHT} = \Sigma u$ (t/m) | - 161 | 42 | 43 | a) 80<br>b) 1,5<br>c) - 7 | 19 | - 30 |
| $U_{LEFT} = \Sigma u$ (t/m) | - 100 | 0 | 8 | a) 19<br>b) 4<br>c) - 15 | 3 | 4 |




a = 99
b = 6
c = [illegible]

FIG. 10 SUMMARY COMPARISON OF BENEFITS TO MASS STABILITY SCHEMATIC, MERELY BOUNDARY U VALUES.

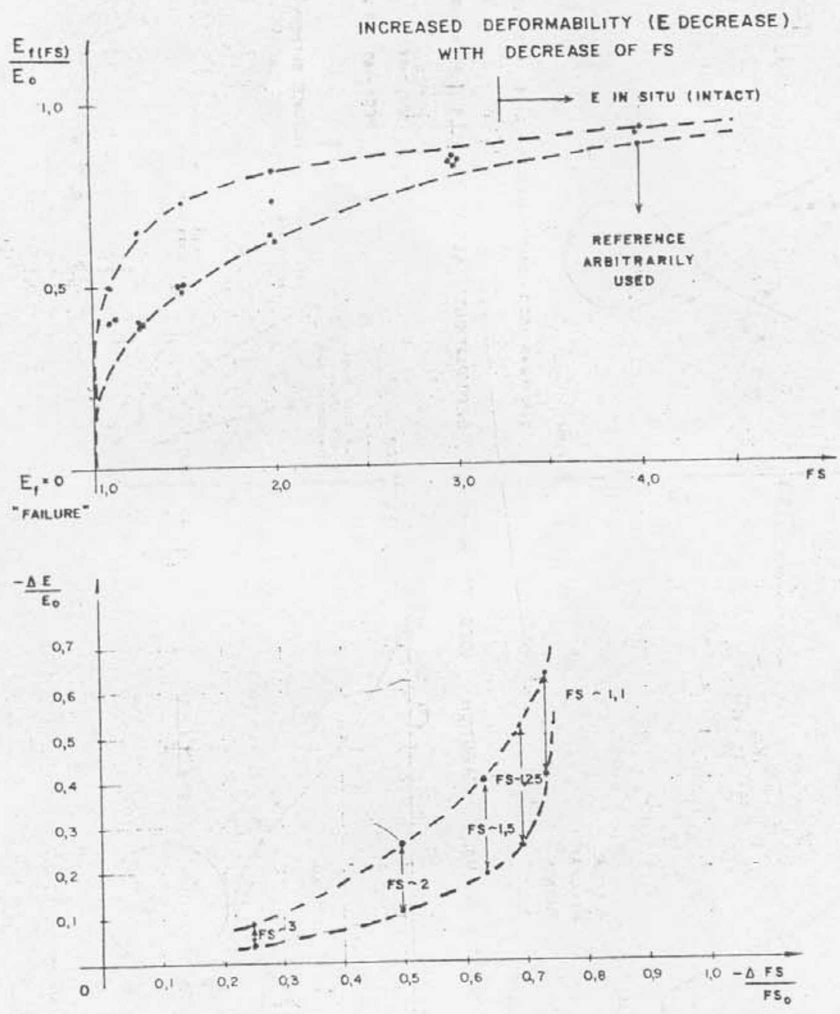



FIG. II TYPICAL TREATMENT OF PLATE LOAD TEST DATA ( LOADING )

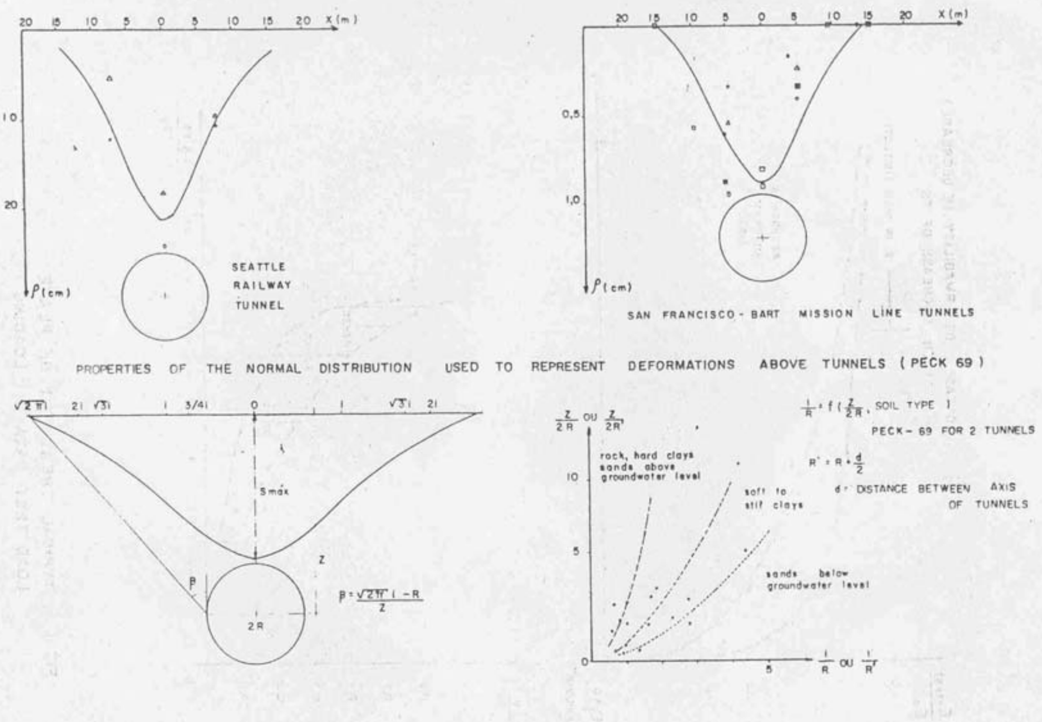



FIG. 12 DEFORMATION IN TUNNELS ( PECK 69 )

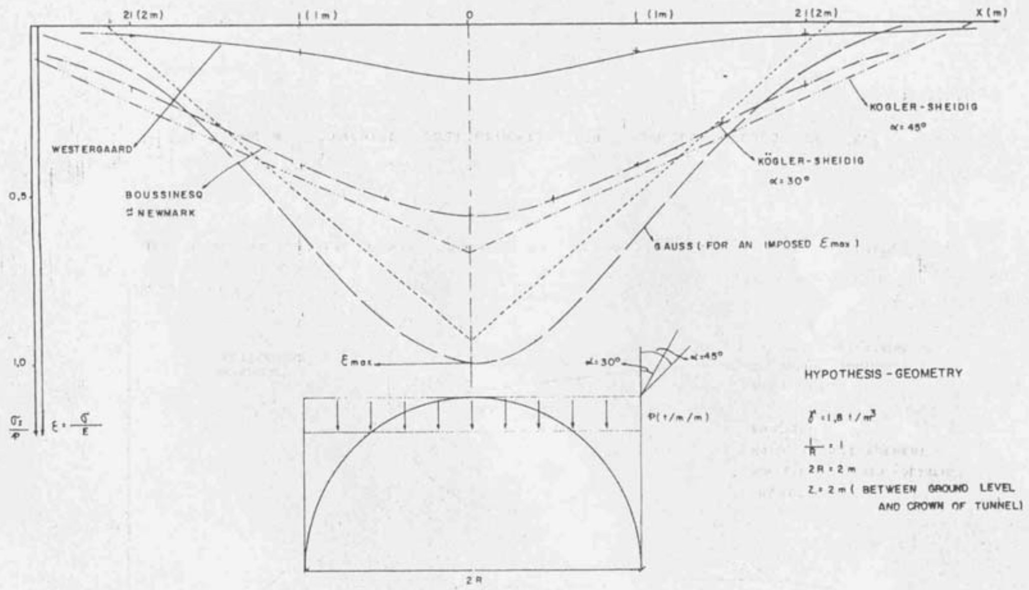
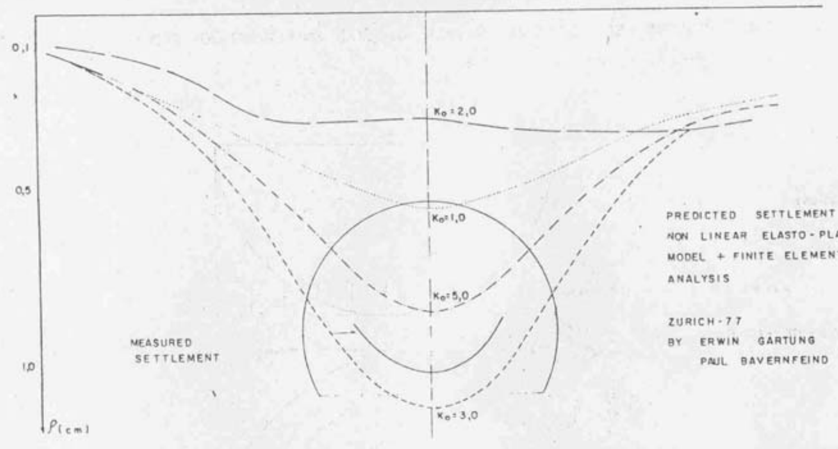



FIG 13 DEFORMATIONS DUE TO STRESS RELEASE VS GAUSSIAN CURVE




SUBWAY TUNNEL AT NURENBERG-GERMANY · STATION NEXT ST LORENZ CHURCH - REFERENCE SECTION

FIG 14 COMPUTED SETTLEMENTS FOR VARIOUS VALUES OF $K_o$

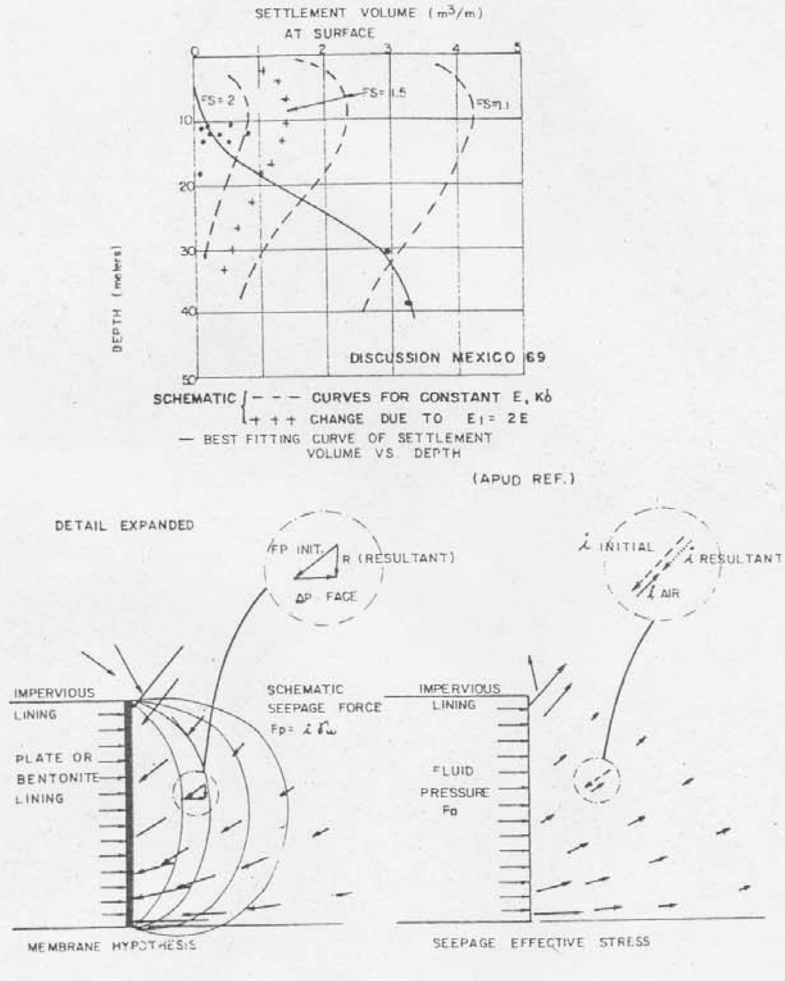



FIG. 15 REEXAMINING QUESTIONS ON TUNNEL SETTLEMENTS