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PECULIARITIES OF "IN SITU" BEHAVIOR OF TROPICAL LATERITIC AND SAPROLITIC SOILS IN THEIR NATURAL CONDITIONS: DAM FOUNDATIONS

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1. INTRODUCTION.

As requested by the Organizing Committee, this General Report considers as a starting point the 6 papers submitted to this International Conference, plus the Final Draft Report, Chapter 3.4 Dam Foundations, under preparation by the ISSMFE Technical Committee as a state-of-the-art 1985 Progress Report (Refs.01, 07). These starting references (and all other) are listed at the end of this report. Since the projects mentioned in those references are modest in number, and the General Reporters were in some or large part associated with each and all of those projects while present experience encompasses also many other dams, both older and contemporary, it has seemed necessary to draw further, from as broad a base as possible, for profitable discussion.

2. DEFINITION OF "TROPICAL RESIDUAL SOILS, LATERITES, AND SAPROLITES (TRSLS SOILS)". IDENTIFICATION AND CHARACTERIZATION.

Although fringe cases may occur wherein opinions would differ, there is an adequate engineering consensus of what soil horizons are meant under the above broad classification. However (or possibly ipso facto) professionals and experts stumble on attempts at more precise definition. One of the reasons is that "tropical weathering and laterization processes" are of wide range, and affect the broadest imaginable range of geologic formations (including sediments). Another glaring one is that the conventional grainsize distribution tests (potentially representing a limiting "unit-particle" condition under maximum deflocculation), and the totally remolded Atterberg Limit tests, both developed for classifying sediments, have been systematically employed in TRSLS mottled, cemented-nucleated and crumb structure interspersed with macropores: in short, one lacks, for the very start, a set of index tests for identification and quantified classification of TRSLS horizons. (e.g. Ref. 16).

The question of identification, test characterization, and possible subdivision of nomenclature, of and within TRSLS soils, pertains to another chapter of the Technical Committee 1985 Progress Report, and lies outside the scope of this General Report and Session. It must only be reminded that those endeavours at appropriate classification must modestly shy away from presumptions at definitive and universal definitions, in order to avoid excessive frustrations. Incidentally, it appears that another strong reason for the confusions may well lie in the fact that significantly differentiable TRSLS soils may behave in essentially consistent manners with regard to routine average geotechnical parameters (e.g. Dib, Ref.03 on oedometer compressibility) while reserving the somewhat more surprising differentiations for less frequently tested parameters. Classificat ions and behavior parameters are not absolute, but aimed at specific purposes: thus it is important to bear in mind the problems and engineering solutions associated with dam foundations, when optimizing index tests for characterizations of TRSLS soils as horizons under prospective dams.

In a pragmatic professional sequence one might discuss the significant dam foundation problems under the stages:- Detection - Investigation - Problem range quantification - Solution options - Monitoring - Corrective action.

Mr. Leme's Chapter (Ref.04) considers a subdivision into groupings: (a) Lateritic Porous Soils (b) Saprolites (c) "Cuirasses" - Lateritic Concretioned layers (d) Biologically worked soils - "canaliculi". Among the four papers, two present preliminary indications regarding presumed inferences of occurrences:-

(1) Cadman and Buosi (Ref.01) insinuate from the Balbina dam that canaliculi occur in "greatest number and concentration by far" in residual soils than in alluvial materials, and "networks of inter-communicating tubular cavities were also found"; from the Samuel dam (of which only about 15km have been yet opened up, out of the 60km length of dikes) that all soil formations have canaliculi "but the clayey alluvial material has shown the greatest concentration of the subvertical canaliculi constructed by earthworms (uncommonly large species) ... to depths of 4 or 5 meters"; and from the Tucuruí dam, that the subvertical canaliculi to great depths (~30m) were dominantly associated with "residual soils of metabasic rock and diabase", wherein "an infinity of small tubular canals ... were found which formed a network of intercommunicating cavities". All canaliculi are stated to occur "always in soils well laterized".

(2) de Sola (Ref. 07) generalizes from the Guri dam that "the immediate consequence of weathering is the loss of unit weight", although surface creep (in steeper slopes) "breaks porous structure and compacts the intervening mass".

(3) Dib (Ref.03) supplies data on variations of chemical and mineralogical compositions (in some soils, and in one soil varying with depth), and accepts at face-value (his Fig. 3) a published arbitrary grouping into non-lateritic soils, lateritic soils, and true laterites: the moot question is whether these subdivisions have any geotechnical significance or might well profit from independent overhaul.

As General Reporters, we earnestly inquire if the principal admonitions at the present state-of-the-art should not be:

(I) The obligation of prudence not to generalize on the vastly varied and varying conditions likely to occur;

(II) international geotechnical emphasis that one of the main differences between sedimentary geotechnique and TRSLS geotechnique is

(II.1) the inexistence of definably distinct horizons for quantified treatment, but rather, a gradually varying continuum (Ref. 20);

(II.2) the significant and frustrating variability of conditions between contigous volumes of soils in a self-same "visual horizon";

(II.3) the need to distinguish clearly between parameters defined through laboratory test variations $\varepsilon = f(\sigma')$, $k = f(\varepsilon)$, $s = f(\sigma')$ etc. for each given block sample, and the corresponding parameter variation that should describe the subsoil's changing parameters with depth and with incremental applied load, in quest for nominal subdivisions into horizons if necessary, or for the quantifiable equations of different geotechnical parameters within the varying continuum. (Ref.20).

Is it fair to generalize any of the "categories" and "occurrence--associations" briefly transcribed above? We strongly doubt it, but the question is of great relevance, and it is offered for debate, firstly to the authors themselves. For instance, may one associate interconnecting canaliculi (only) with saprolites of metabasic rocks and diabases? The test question is focussed by inserting the word "only" as shown within brackets. Or else, by making the opposite question, e.g. "therefore, may one rest reasonably assured that interconnecting canaliculi will not be due to other than biologically (termite) worked horizons, and need not be feared in other than the metabasic or basic saprolites?"

Possibly the most difficult problem in TRSLS soils is for the geotechnical engineer to be able to circumscribe what identifications and classifications guarantee him the strongly probable occurrence-associations and the reasonably confident right to exclude certain problems. Since in theory all problems may occur in any given foundation, and the engineer's approach is to employ broad classification in order to limit as rapidly as possible his range of worries, we emphasize that the most momentous problem in TRSLS soils is to identify and classify in a purposeful and efficient manner.

In a sedimentary sand we promptly acquire the right to set aside questions regarding plasticity, imperviousness, cohesion, shrinkage, etc. How do we stand with TRSLS horizons once identified?

One cannot generalize on weathered soil horizons except in a highly simplified and idealized condition. Each weathered horizon must, by definition, be associated with the macro-structure (joints, lenses, bands, etc..) of the respective parent rock (geology) and also with the respective microstructure and pore structure defined by petrography, mineralogical composition, and so on, plus the historically varying conceivable physical, chemical and biological weathering agents at selective attack on these differently constituted volumes, side by side. Thus, whereas much of modern conventional soil mechanics has swerved away from geology (which is a gross mistake and unfortunate), in TRSLS soils the exasperation is that geology is important, sine qua non, but, in its own conventional tools and uses, far from sufficient.

The starting problem in geotechnical engineering is to be able to rely on adequate clues to suspect, expect, and detect, the presence of a given problem, and its degree (relative to the accept-reject decision threshold). Almost every case history published continues to be exasperating because, on closer analysis, it is an "after--the-fact report". We neither want to be caught shutting the stable door after the horse has run out, nor be induced into shutting all stable doors in such hypothetical anticipation that the horses start off by being shut out. Once a problem has been well enough exposed, we can be reasonably confident of being able to cope with it.

HYPOTHETICAL THEORIZATION FOR TRSLS GEOTECHNICAL ENGINEERING.

An attempt at conceptual generalization for residual soils and saprolites was submitted by the senior author (Hong Kong 1972, Ref. 20), wherein such soils might be reasoned as the product of "natural selection" under cumulative micro-attacks by all possible weathering agents, whereupon the "weakest volumes" are plucked off, leaving the remaining "structure" responsible for "carrying the load". Corrosive ecological action from a stronger status (e.g. rock or indurated sediment) leaves a material with a histogram of quality always better than the minimum necessary (cf. Fig. 1 as an analogous example). In such a condition, although on an average the overburden effective stress $\gamma' z$ applies at a given depth, it would automatically follow that the overburden stresses would vary greatly along the plane, being much higher than average on rigid volumes and much lower than average on soft pockets: in fact, pockets are "allowed to soften" (often reaching liquid limit condition, and often being totally removed, forming micro-cavities or canaliculi) only to the extent that the surrounding rigid body structures assume the responsibility of carrying loads and shear stresses.

It was concomitantly postulated (1972) that inasfar as even sediments must have some dispersion (although much less, being developed from a homogenized suspension and slurry condition), in conventional sediment soil mechanics no soil element need be any stronger than as required to carry (be consolidated under) its share of the overburden stress: therefore, it is not merely by prudence, but by statistical mental modelling that one should recognize that in such sediments the weakest soil elements do play an important role in overall soil behavior. Such postulations would require development of statistical confirmatory or revisionary data, and might have generated distinct sets of statistical working rules for saprolite vs. sediment geotechnical engineering.

Since the introduction of statistical reasoning within the reality of natural selection appears to have been (and continue to be) notoriously undigestible to the conventional geotechnician, we take the liberty to present Fig. 1 merely as an illustration. It reproduces compaction data from the Tres Marias Dam (Ref. 27, Fig. 5) on the effect of the self-same specifications, and the consequent histograms of real conditions, resulting from truncation either on the wet side, or on the dry side. Can anything be more obvious?

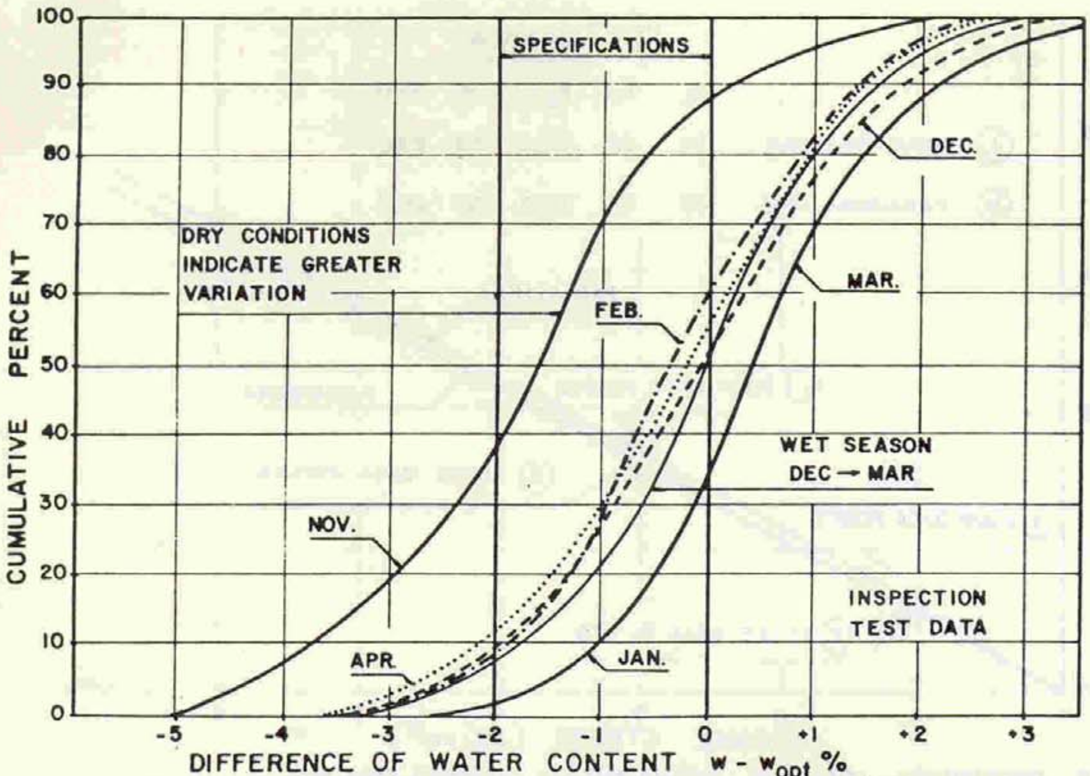


FIG. 1 SPECIFICATIONS FOR WATER CONTENT AND CONSEQUENCES OF TRUNCATION AT UPPER vs. LOWER LIMIT. INCLUDE CONSEQUENCES OF TEST ERRORS (APUD DRW. 5, REF 22)

Almost nothing had been done until recently. The profile test data reported on a residual soil from gneiss in a 8m deep test and sampling pit (Ref.10), hand-excavated without confinement, doubtless incorporate adulterations due to stress-releases in pitting and sampling. Meanwhile the invaluable results derived by the theory of weathering modelled as a weakening process (Ref.33) necessarily begin by referring to average homogeneous-mass conditions. With regard to statistical variabilities intrinsic to the soils, and the feel of which may be the soil elements more dominant in any behavior, the very concept has been left dormant. If in sediments the K'_o conditions (especially long-term) have been left sorely uninvestigated, in saprolites both the $\gamma' z$ variability and the K'_o conditions are recognizedly unknown.

In a similar vein within a saprolite grainsize distribution (statistically considered) there is some inescapable influence based on differentiated dominance of mineralogical strengths of the grains. Thus a special investigation into residual ϕ' values in compacted saprolites of granito-gneisses of the Paraitinga and Paraibuna dams (Ref.31) because of apparently high mica contents, revealed that ϕ'_r were "extremely high" (Fig. 2; probably because of angular quartz grains), although "surprisingly, the Paraibuna material (with) a lower liquid limit, plasticity index, and minus 2μ content ... exhibited a lower ϕ'_r value ... possibly ... attributed to mineralogical differences (perhaps a higher mica content)". These results had been considered quite foreseeable by

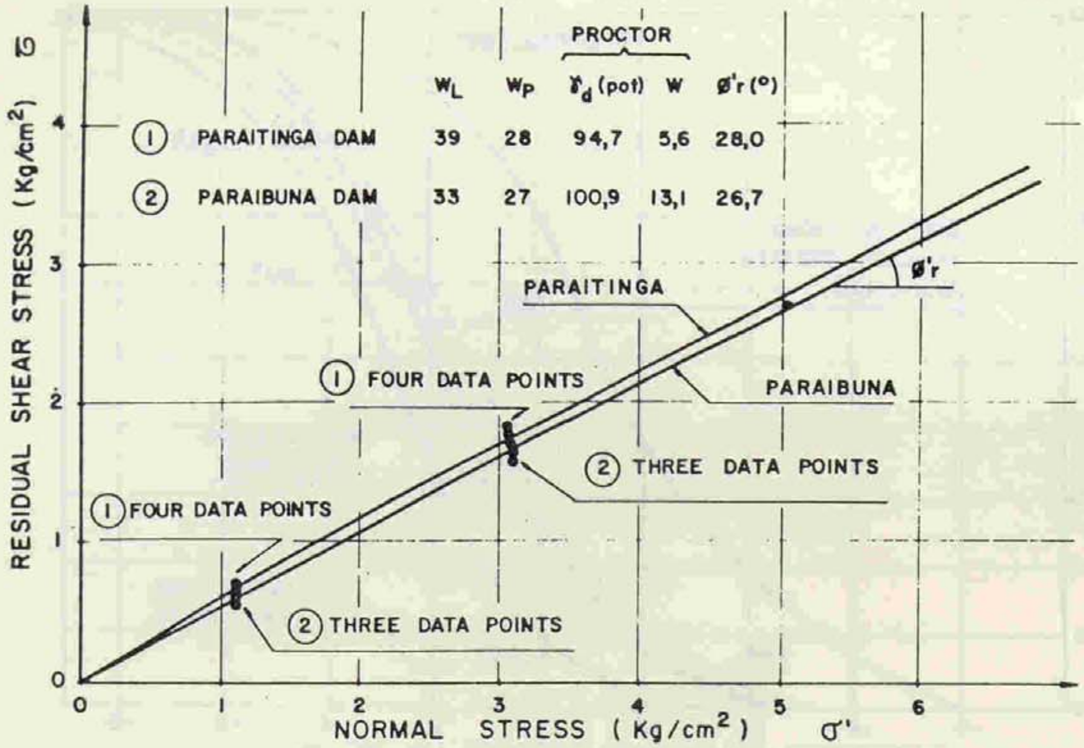


FIG. 2 RESIDUAL SHEAR STRENGTH ENVELOPES

ourselves.

Within this panorama we should postulate that laterites present a further degree of total ignorance. Laterization processes of "removal" of certain volumes and concomitant induration of others, start either on sediments or on saprolites, conceptually different: but the chemical induration processes and the entymological or botanical removal processes have no reason to respect a hypothetical statistically static equilibrium around the need to support overburden stresses. Induration can be to lithological strengths capable of carrying high structural loading (laterite "soft-rock" masonry, limonite concretion cuirasses that offer absolute refusal to precast pile driving, etc...), and macro-pore generation can reach extremes associated with highly collapsive soil structures. It is difficult to progress in a technology without some hypothesis, some mental postulation: however mad the start might seem, there must be a "method to our madness".

Since there is not even a candid recognition of such a need, let us turn to typically recognized problems and dam foundation solutions, as associated with after-the-fact descriptive case histories.

For the foundations of embankment dams one must generally consider principally problems of shear strength (as affecting bearing capacity and slope stability), problems of foundation deformations (settlement, heave, collapse), problems of permeability and possible retrogressive internal erosions of piping, and possible problems of chemical, bacterial and eventual animal activity in long-range reservoir operation, and finally the problem of saturation and eventual dynamic instability under seismic activity. The principal construction problems are associated with foundation preparation and cleanup for support of the dam structure, and foundation subsoil treatments.

Foundations of concrete gravity or buttress dams (cf. Ref. 24) in principle follow the same reasonings although requiring better support conditions for technical and economic compatibility. There appears to be some implicit hint that concrete gravity dams cannot be founded on weathered TRSLS horizons, but such a hypothesis can be proven false both by theory and by reference to many a satisfactory case history of professional practice. The requirements are more severe and with narrower margins of tolerance, but can well bemet with the appropriate testing, design, and construction control. The following comments will be limited to embankment dams because they involve the higher and more important dams of modern times, and because the papers under review have been limited to such a scope.

The following discussion would hopefully employ a rational subdivision, with references to the papers wherever appropriate.

4. FOUNDATION SHEAR STRENGTH, BEARING CAPACITY, AND SLIDING STABILITY.

The question of foundation shear strength in TRSLS soils has received attention in scattered projects, and has generally been situated in two extremes: in a vast majority of cases the trend has been to assume or conclude that "no problem exists at all (provided stripping is lowered to adequate horizons)", and in a few cases extreme concerns have been raised because of conventional tests coupled with presumed analyses.

4.1. Adequate bearing condition.

The problem of supporting quality of soil foundations for embankment dams has not been raised with regard to conventional "bearing capacity" failure hypotheses and formulae. The supporting area is too wide (the formulae lead to big increases of failure pressures with increased dimensions of loaded areas); the hypothetical vertical loading (trapezoidal) on the foundation plane is not a soft load, but suffers very significant redistributions due to the internal shear stresses within the embankment thickness itself, simultaneously acting as the loading and as a resisting element along any critical plane; the increases of foundation strength with depth ($\gamma' z$ effect coupled with decreasing weathering, cf. Fig. 27, Ref. 20) and frequent variations along the supporting base are too significant to lend meaning to uses of "bearing capacity concepts or formulae" of footings or small-dimension tanks or rafts. The foundation shear failure hypothesis is treated via sliding analyses.

The two indices for estimating foundation competence that are most frequently used are SPT penetration resistances and the in situ dry densities compared with the respective Standard Proctor maxima via the percent compaction PC%. The designer must be emphatically warned against the error and danger of using such SPT and/or PC% indices in saprolites of laminated rocks which may exhibit high anisotropy of shear strengths normal and parallel to the laminations. If a moderately isotropic soil mass is at stake the rough relationships that inexorably connect in situ dry density (PC%) with SPT and with

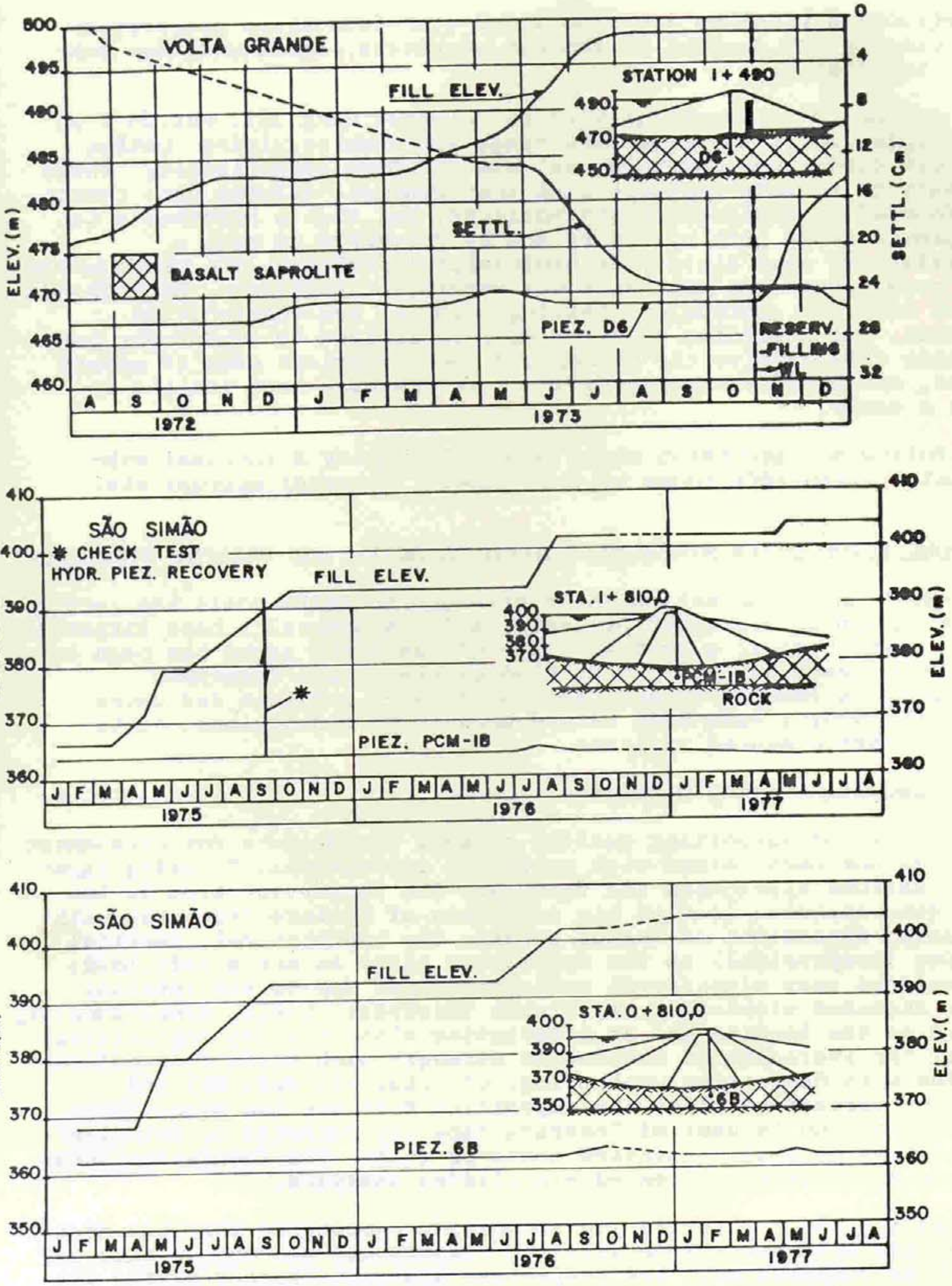


FIG. 3 VERY LOW $U_{CONST.}$ IN LOW DENSITY WEATHERED BASALTS

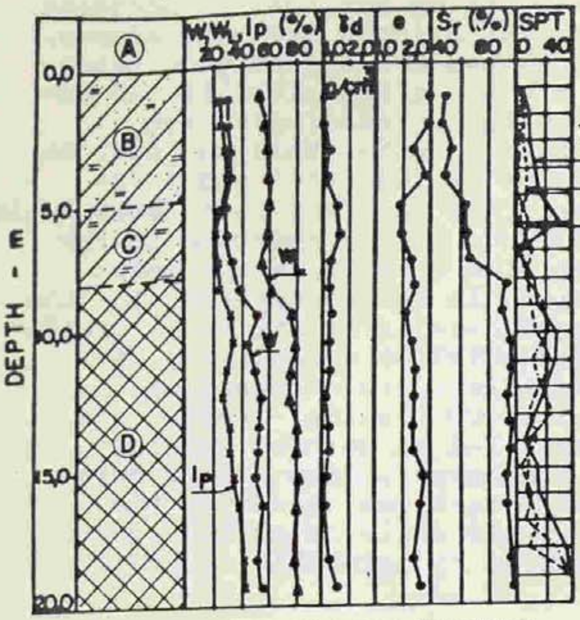
shear strength should serve for establishing horizons of adequate support for embankment dams of different heights and side slopes. One important proviso is that both PC% and SPT can only be reasoable indices of in situ strengths, and the difficulty lies in extrapolating to estimated behaviours under increasingly high embankment loads, i.e. estimating the $\phi = ds/d\sigma$. This can only be done via estimates or determinations of S% (in situ) and $\Delta\varepsilon/\Delta\sigma$ compressibility. The great boon comes from the fact that TRSLS soils generally have low S%, some in situ suction, and moderate to low compressibilities; and also that the slow rise of embankments generally guarantees drained conditions in the foundation. In the foundations of high porosity weathered basalts of the Volta Grande ($\simeq$ 50m) and São Simão ($\simeq$ 100m) dams specialist consultants of greatest authority expressed considerable concern regarding foundation supporting capacity based on UU triaxial tests, and constant volume piezometers were installed to furnish warning: however, these revealed no perceptible construction period pore pressures (Fig. 3), confirming local intuitions, based on the macropores and air-pores typical of TRSLS soils being the controlling boon for dissipating pore pressures by permeability and air--pore compressions.

None of the 6 papers, nor the "Dam Foundation" chapter, broach the subject of adequate shear bearing horizon in TRSLS soils. As General Reporters we submit the observation that high granular stockpiles in construction sites and mine wastepiles have been generally dumped on TRSLS surface horizons without selection or preparation of supporting conditions. Would any colleague advance field indications on case histories evincing such base failures? At present it would seem that under normal conditions a drained behavior would apply, and bearing capacity failure conditions result rare.

At any rate, the question is of considerable importance regarding the requirements of foundation excavations under upstream and downstream "shells", and thus the question is submitted to debate with the following preliminary comments.

. Very many papers have summarized the typical profiles of the highly weathered tropical horizons, including (a) porous red clays (weathered in situ, either from sediments or representing the most advanced attack of the underlying saprolites; often difficult to distinguish from elluvial horizons); (b) compressible low-density saprolites (potentially clayey of high plasticity); (c) saprolites of gradually increasing density, incompressibility and strength; (d) down to decomposed rock and sound rock.

Merely for the purpose of refreshing the reader's memory on some of the foundation conditions that, in their days, gave reasons for pertinent concerns, we reproduce in Fig. 4.1 some typical subsoil profile data from the Itumbiara dam (110m, Ref. 25), in Fig. 4.2 some of the typical SPT data from that dam's foundations showing the great dispersions which any index tests present in such soils, and in Fig. 4.3 some of the data from the porous red clays of the Tres Marias Dam (1957-61) that was the first one widely publicized as facing the problem of very compressible porous clays in an extreme condition. (N.B. It might be of historical interest to note that the question of construction period stability was not even raised in the minds of the consultants, dominantly international, on the occasion).



Ⓐ PROFILE PIT PE-94 LEFT ABUTMENT
Ⓑ RED SILTY CLAY W. FINE SAND (ELUVIAL)
Ⓒ RED SILTY CLAY W. LATER. CONCRETIONS
Ⓓ MOTTLED SILTY-CLAY (BASALT SAPROLITE)

FIG. 4.1 TYPICAL TRSLS PROFILE OF BASALT FOUNDATION ITUMBIARA DAM (APUD FIG. 4, REF. 24)

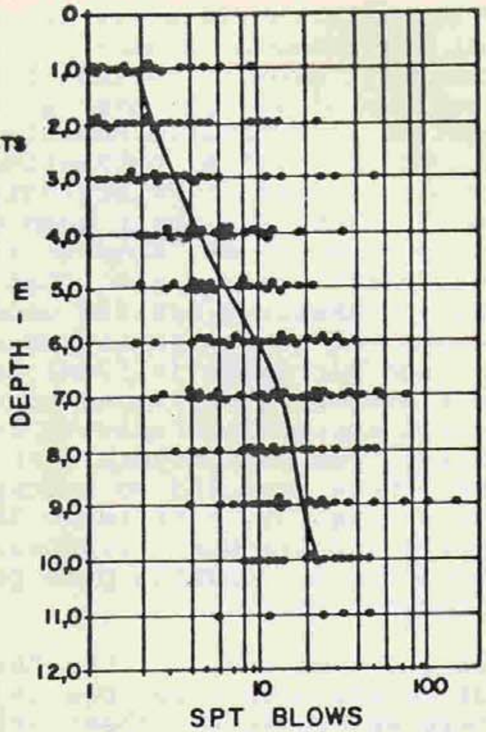


FIG. 4.2 VERY WIDE SCATTER (APUD FIG. 5, REF. 24)

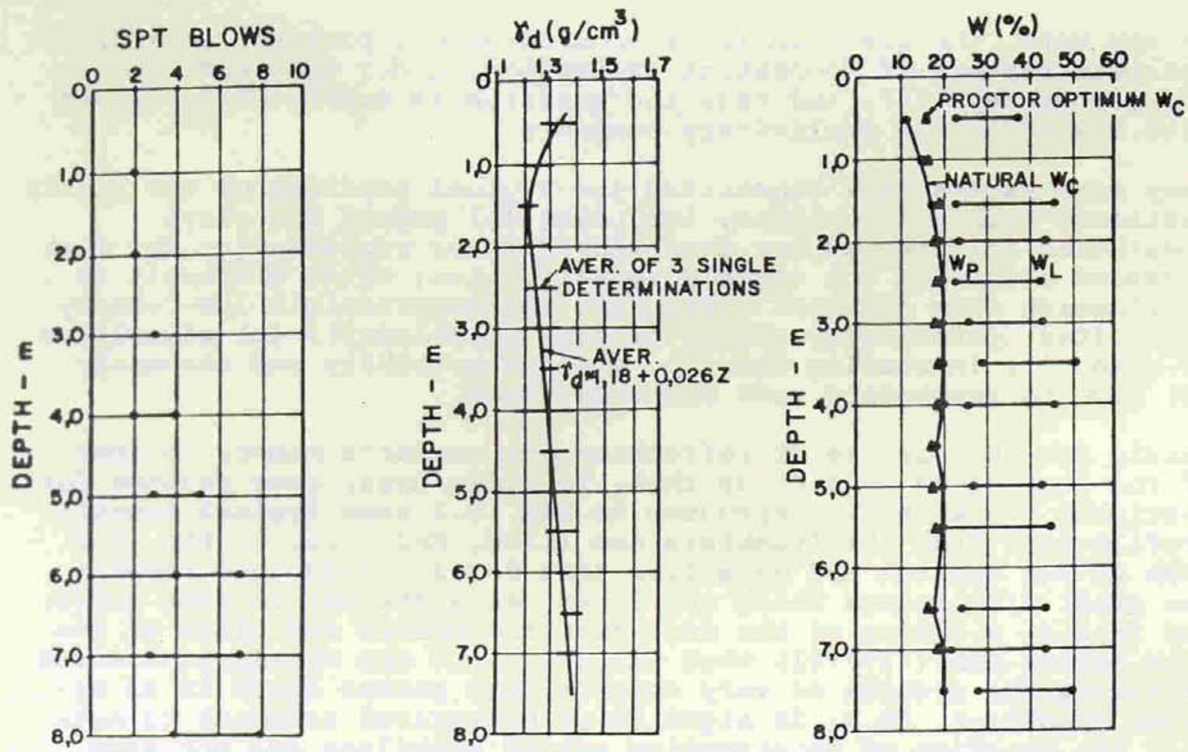


FIG. 4.3 POROUS CLAY - AVERAGE DATA FROM 3 TEST-PITS (APUD FIG. 2, REF 27)

Using such profiles as hypothetically representative, one may compare some of the intuitive rule-of-thumb indices that have been used in preliminary estimates; and, in immediate consequence, one may propose some rough semi-theorizable justifications for the relatively favourable bearing capacity behaviours of embankment loadings on TRSLS horizons.

As a first engineering estimate we have prepared the sample graphs of Figs. 5.1 and 5.2 using the nomographs of elastic solutions published in Ref. 26 (pages 212, 213). These are for embankment on elastic foundation, foundation assumed weightless, and only the effect of the embankment considered: i.e., therefore, the incremental effect on the elastic foundation is represented. The graphs are limited to the cases of embankment slopes of 45°, 30° and 15° (slopes varying between 1:1 and IV:3.73H that cover essentially all practical cases) because of the available published nomographs: intermediate cases could be interpolated and extended, if and as desired; the present intent is principally to establish a modus faciendi. The basic intent is to estimate, for various representative points within an upper horizon of soil, what would be the estimated obliquities of stresses, and how these would vary during construction progress, and, in order of magnitude, what would be the critical obliquities reached. Of course, everything depends on the initial state of stresses of the in situ soil element, which is probably the least known important parameter in sedimentary deposits, and altogether unknown in TRSLS soil horizons.

In Fig. 5.1 we have postulated the embankment rising in horizontal layers, with constant external slopes, and have estimated the stress-obliquities for heights of fill equivalent to 60%, 85% and 100% of maximum. For each slope investigated, 45°, 30°, 15°, we have plotted two stress-obliquity lines at angles α_1 and α_2 at nominal factors of safety FS = 1.3. Firstly it is postulated that one may accept a reasonably linear-elastic behavior in soils up to FS $\lessapprox$ 1.3. Thereupon, by showing that the plotted stress-obliquities determined generally lie below the α_2 line, we can satisfy ourselves that elastic base conditions are not, in general, forced to face stress-obliquities as those of the slope itself.

Moreover, if a slope is built satisfactorily stable, it doubtless has a minimum FS > 1.3. Therefore, for each of the slopes surely the strengths available in the embankment must lie above the α_1 lines. As a corollary one further deduces that there is a significant reserve strength from the fill itself (above α_1) to compensate for any local plastifications (base condition approaching α_2).

In Fig. 5.2 we have repeated the computations (retaining the "elastic" assumption even for end-dumped advance) for a fill advancing with constant slope and significant height. The analysis is made on a hypothetical fill extended to the farthest position 3 and, for the various points A, B, C, D, E, using internal positions 1, 2 as hypothetical transitory situations for an advancing front 1-2-3, computing the predictable changes of stress obliquites. The results are plotted in a manner similar to that of Fig. 5.1 and with analogous proposed method of interpretation. One intuitively recognizes that such a condition (used in stockpiles) embodies a favorable "pretesting" of the foundation regarding shear stress

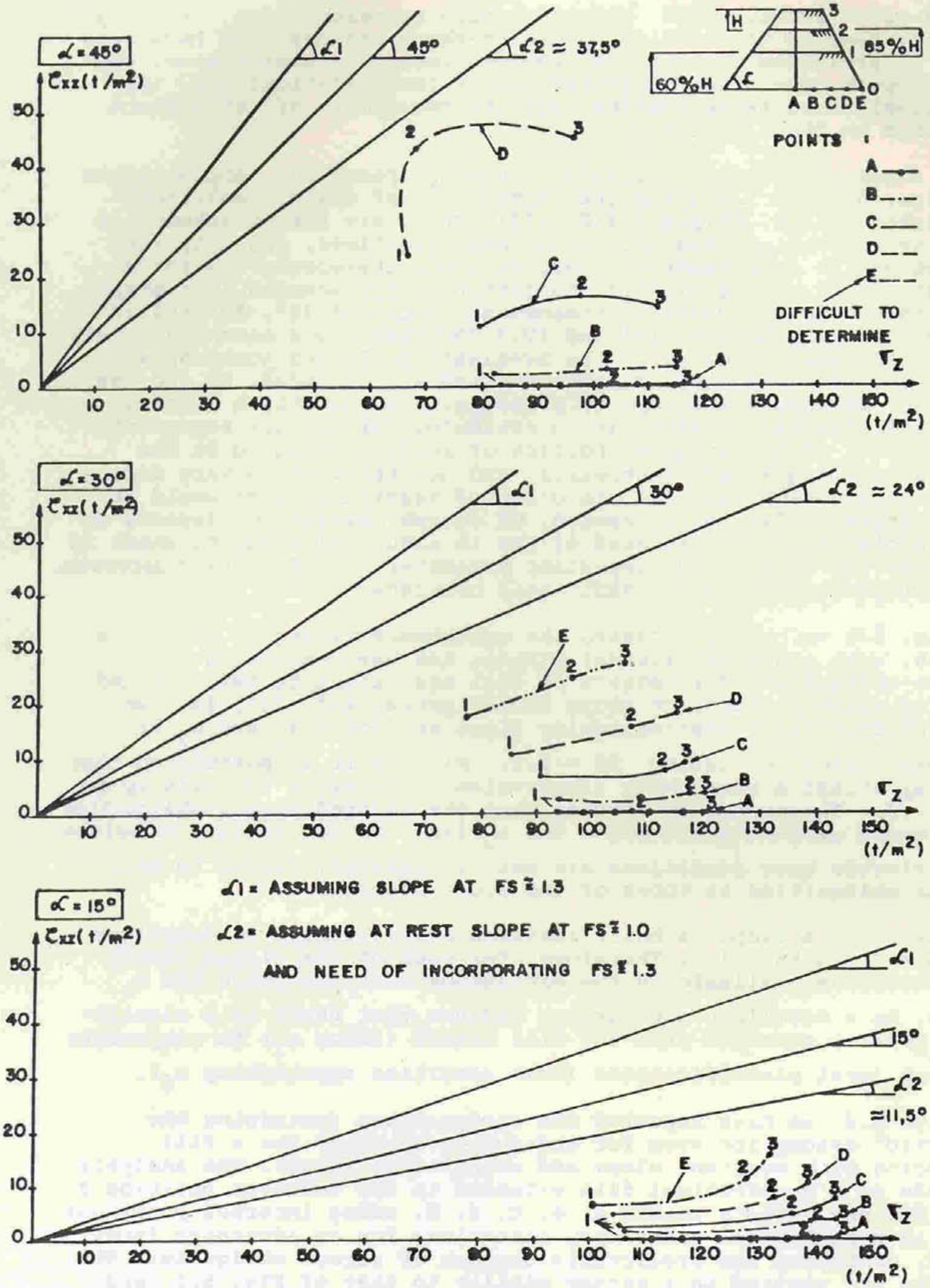


FIG. 5.1 UNDER RISING EMBANKMENT

FIG. 5 FOUNDATION STRESS OBLIQUITIES

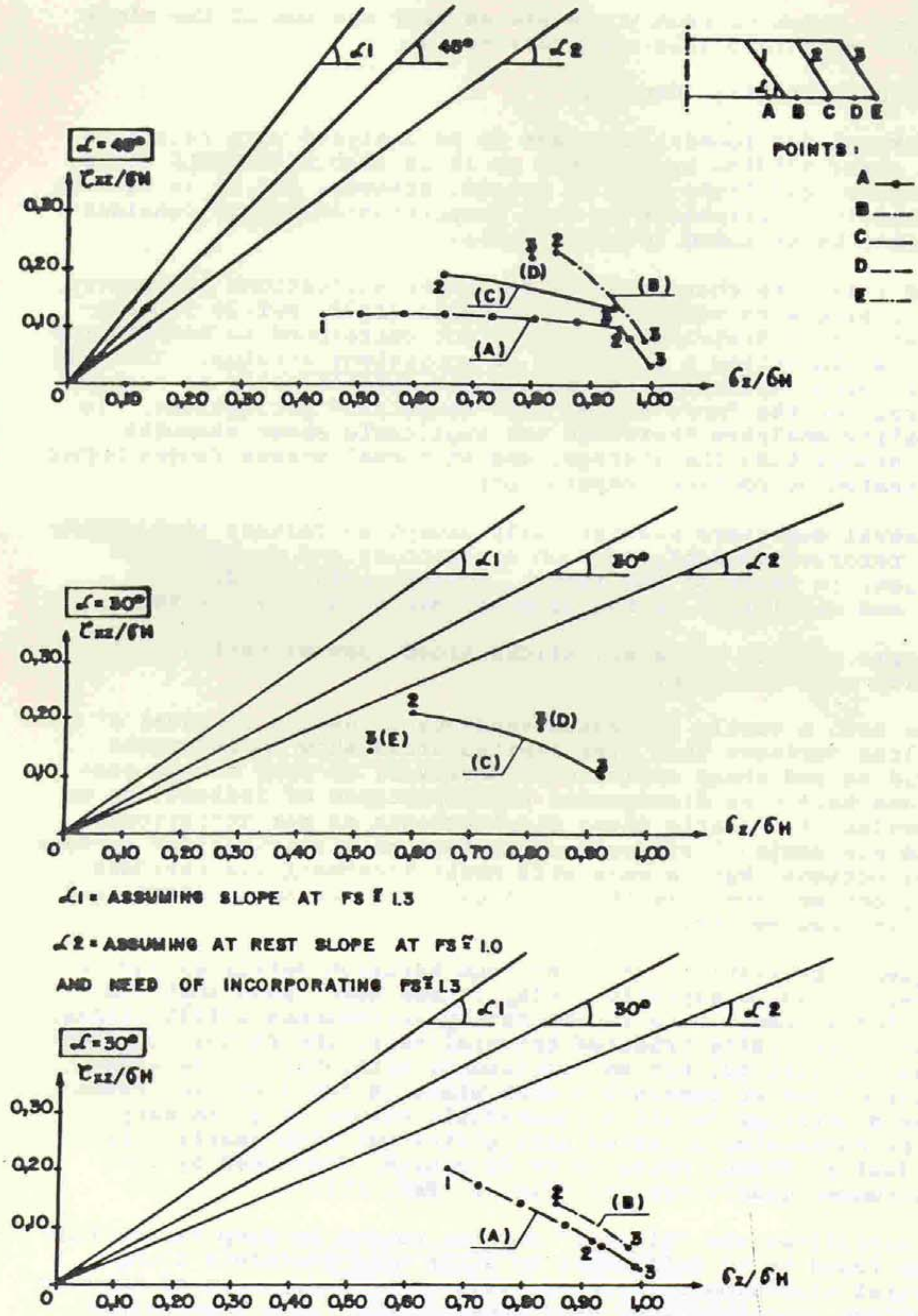


FIG. 5.2 EMBANKMENT OF ADVANCING END-SLOPE FRONT

FIG. 5 FOUNDATION STRESS OBLIQUITIES

obliquities, which go from worse states near the toe of the slope to the much more favorable ones further in.

4.2. Sliding stability. General

All embankment dam foundations have to be analysed with regard to critical shear sliding hypotheses: so it is also with TRSLS soils. Some important questions must be raised, however, and it is not the purpose herein to visualize in what proportion analogous considerations might be extended to other soils.

(a) TRSLS soils are characterized by harder nucleations interspersed side by side with softer ones. De Mello (1972, Ref.20) postulated that stress distributions would not correspond to homogeneous media but would follow a principle of equivalent strains. Thus, incremental normal and shear stresses would automatically be higher than average on the "more rigid, more resistant" nucleations. In any stability analyses therefore the applicable shear strength would be higher than the average, and at normal stress ranges higher than indicated by routine computations.

These General Reporters realistically accept as certain that nobody has ever resorted to applying such a reasoning and consequent calculation. In favor of the future, however belated, direct comments and criticisms on the proposal are earnestly invited.

(b) Historic sliding surfaces, slickensided. Use of residual ϕ' values with zero cohesion.

There has been a vastly increased tendency to assume residual ϕ' values along surfaces that over limited areas show undisputable slickensiding and shear striations. Moreover, in many a case geotechnicians have even disregarded the importance of indications on the direction of historic shear displacements as per striations, and would use residual strength parameters on a slickensided surface in all directions. Many a case will merit treatment via residual strength, but we emphasize that each case must be scrutinized, and stand on its own merits.

Two frequent observations are mentioned herewith merely as sample reminders. In many a saprolitic clay it has been found that old slickensided surfaces were recemented by ferruginous infiltrations, whereupon in specially oriented triaxial tests the failure surface developed parallel to, but not coincident with, the slickensided plane (which however remained a weak plane in tension). In cases where the mineralogy is not homogeneously clayey (e.g. in many saprolites containing oriented mica plates but also quartz grains) the residual ϕ' values prove to be very high, dominated by the by the stronger quartz grains. (Fig. 2, Ref. 31).

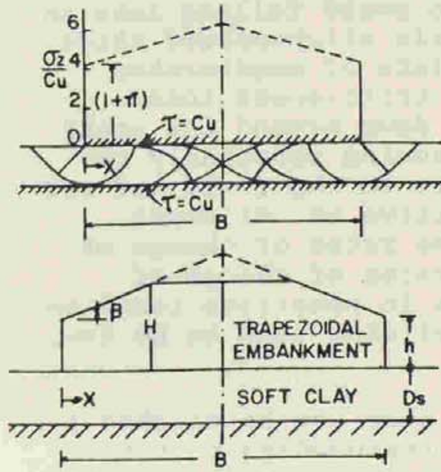
In many saprolites the "elastic" strains caused by deep excavations have been found to be sufficient to align mica platelets along preferential mineralogical planes, giving the impression of historic slide surface slickensiding: upon opening test pits some meters back of the excavated face, where stress-relief strains are much smaller, one often confirms that the presumed plane is not continuous. The fact that movements of but few centimeters (thousands of times the particle-size) are sufficient to align the platelets (especially under higher normal stresses) is undisputable, and the evidence is ipso facto more accentuated near surface: one must, therefore, investigate carefully in order to avoid falling into an automatic pessimistic conclusion of a historic slickensided slide surface of big dimensions. Thereupon, the risks of accelerated failure must consider the estimated revised brittleness index (comparing $ds/d\varepsilon$ up to the peak vs. $-ds/d\varepsilon$ down beyond the peak) as proposed by de Mello (1977, Ref. 19) (assuming reasonably representative stress-strain curves, applicable to the field and not merely to tested specimens), and any contractive vs. dilatant tendencies. In our experience, since the time rates of change of causative factors tend to be slow, and the rates of change of effects far from rapid (for instance, not as in sensitive Canadian and Scandinavian clays), cases of dramatic sliding tend to be few, under special conditions only.

Another foundation plastification condition that can be checked for rough very conservative estimates is that corresponding to the squeezing condition (Jürgenson, 1984, Ref. 11, etc.) of a presumed soft "clay" ($\phi = 0^0$ material) of limited thickness (cf. Ref.30 p.225 for instance). By reference to extreme cases of $\phi = 0^0$ saturated sedimentary clays, and ($c = 0$) drained ϕ' pure sands, we can demonstrate that TRSLS (c, ϕ, unsaturated) soils do not present bearing capacity problems.

Firstly one estimates average overburden stresses down the profile, since at least some nominal stress values must be used for calculations, and at least in closed-cycle correlations they fend off too much criticism. Without yet enough data for satisfactory double regressions, we may tentatively report having often successfully adjusted equations of the types (a) $\gamma \approx 1.4 + 0.035$ SPT (t/m^3), (b) $\gamma \approx 1.4 + 0.031\, z$ (t/m^3) for depths z, m and $1 < \text{SPT} < 8$ in upper maturely weathered tropical horizons. Such relations may be used for estimating overburden stresses, and thus, by appropriate estimates of u (including suction, if and where applicable), may estimate average nominal effective vertical stresses and presumed consolidation stresses.

Although TRSLS soils mostly exhibit (c, ϕ) behaviors, the very conservative estimates can be made by assuming that the s_u in situ (s_{uis}) is the least shear strength developable in a dynamic loading condition (e.g. in SPT blowcounts) or in "instantaneous undrained embankment loading hypothesis", as regards $ds/d\sigma$. For insensitive clays one has accepted a rough estimate $q_u \approx \frac{SPT}{8}$ kg/cm^2 (de Mello, Ref. 21) which corresponds to $c_u \approx s_{uis} \approx SPT/1.6$ (t/m^2). Estimates of OCR etc. may be formulated without much error since in our experience the ferruginous cations and mostly kaolinitic and inactive clay mineralogies tend to guarantee $\phi' \approx 25 - 30^0$ and $\phi_{app} \gtrsim 20^0$. Thus, with any given SPT profile one can estimate minimum values of c_u for hypothetical $\phi = 0$ loaded clays, and for the foundation stability analysis as per Fig. 6.1.

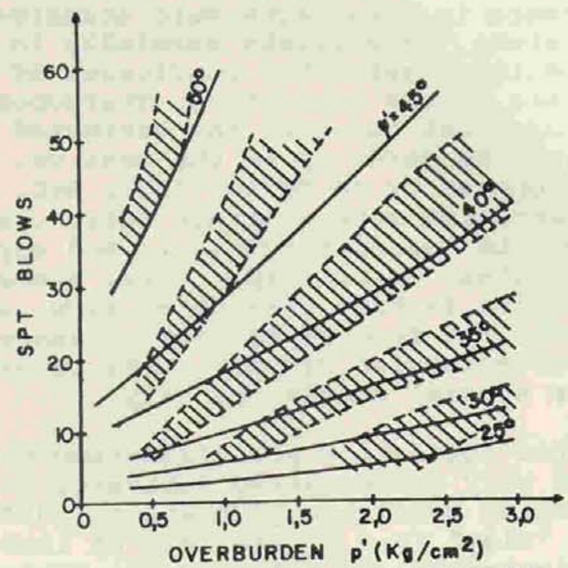
At the other extréme, if we interpret the TRSLS soils under the realistic fully drained conditions of pure sands, but very conservatively omitting the cohesion intercept (i.e. the available shear strength at zero overburden stress, due to preconsolidations, cementations, suction, etc..) we can extract another set of minimum



$$FS = \left[\frac{1+\pi}{H} + \frac{\lambda (H-h) \cot g\, \beta}{H D_s} \right] Cu$$

ADOPTING $\lambda = 2$ AND $\gamma = 2\ t/m^3$

FIG. 6.1 EXTREME CONSERVATIVE PLASTIFICATION, OF SOFT CLAY UNDER TRAPEZOIDAL EMBANKMENT (APUD SILVESTRI, 1983).



CORRELATION WITH CPT q_c: $N = q_c/3{,}5$; $N = q_c/5$

ASSUMING $3{,}5N \leq q_c \leq 5N$

DATA FROM LUNNE, KLEVEN (1982) – CPT q_c

CHART FROM DE MELLO 1971 SIMPLIFIED BY SCHMERTMANN 1975

FIG. 6.2 CURRENT INTERPRETATIONS OF DRAINED φ' PROFILES IN PURE SANDS, USING SPT OR CPT.

s_{is} values from the chart of Fig. 6.2 (de Mello, Ref.21).

In either case it is from such reasonings that simple PRESCRIPTIONS have proved satisfactory for establishing minimum required SPT values for suitable bearing level in TRSLS profiles. For instance, for side slopes around 1V : 2H to 1V : 2.5H an oft-used prescription is SPT > H/5m where H = height of dam above the section. Different slopes, steeper or flatter, obviously suggest some adjustment of this first-order prescription (de Mello, Ref.14).

Comments from the Authors and Panelists are earnestly requested because the depth of excavations to adequate supporting quality are a very significant item of technical, economic, and logistic importance.

In short, as regards adequate bearing condition, first tested under construction period loading, one finds a general acceptance that TRSLS horizons have not presented problems. It is most unfortunate that the internationally current investigation techniques for improved multiple profiling (e.g. minimum of 3 simultaneous equations required for strength, assuming influence of significant unknowns c', φ' and σ') have not yet been introduced in TRSLS soils. The more common established penetration devices could be mentioned as, at least, the Friction Ratio FR cone, and the CPTU. The General Reporters earnestly invite comments.

Leme (Ref. 04) furnishes data on very low $\phi'_r = 10^0$ (expectable) in the high plasticity basalt saprolites, and mentions the case of the

Agua Vermelha right abutment dam in which stability analyses and some inclinometer displacements along a specific subhorizontal plane led to the use of a stabilizing toe berm. Engineering prudence in the face of insufficient data justified the decision, which the Senior Author accompanied as a consultant. However, these General Reporters would invite pursuing the question for the sake of future cases. Principal issues at stake are:-

(1) The comparative brittleness (stress-strain σ, ε curves, cf. de Mello, Ref. 19) between: (a) highly compacted tough clay fill and residual ϕ'_r clay saprolite plane; (b) low stress ranges (tension cracking at the top of the fill) and high stress ranges on the foundation plane; (c) rates of change $ds/\partial\varepsilon$ before and after "failure".

Correspondingly the rates of stabilizing consequence, between removal of weight at the top, and adding weight (for consequent incremental strength) at the bottom, can be very different indeed.

(2) The presumed lack of information on the displacement values (few cms., or much more?) at which the inclinometer observations could be interpreted as establishing thresholds of concern. The very "brittle" behavior of the Carsington Dam failure stands shockingly fresh in our minds. Comments are earnestly requested. Leme (Ref. 04) well emphasizes the importance of compatibilities of stress-strain trends between fill and foundation.

(3) Choice of realistic failure surface.
Very seldom is a circular sliding surface realistic. In TRSLS soils dominated by geologic anisotropies and weakness planes, obviously there is a strong tendency for such a plane to be the sliding surface for most of the slide volume. In those not dominated by geologic weakness discontinuities the typical in situ strength profile increases with depth more than would be derived from test Mohr envelopes $ds/d\sigma$ (Fig. 27, Ref. 20). The much greater brittleness at low stresses and in tension tends to impose a typical progression of failure, first by tension cracking at the top, then followed by bulging at the bottom, and finally by full development of shear sliding along surfaces of shapes composite of deep subvertical and long subhorizontal stretches with a curved intermediate transition.

(4) Miscellaneous.
It lies beyond the present scope to repeat that not only in TRSLS soils but in all other soils too, there are no simple standard rules for details of sliding stability calculation, except within the broad general principles. Therefore, comments are invited on documented peculiarities regarding cases involving TRSLS soils.

4.3. Construction period sliding stability.

The Senior Author has put forth some principles of slope stability assessment in embankment dams (de Mello, Ref. 18) assuming rock foundations. It was emphasized that one does not really analyze stability, "starting from scratch (isotropic)", but ("changes of conditions") changes of stability and rates of such changes with rate of change of the unstabilizing agent (rise of fill). Moreover, the choice of appropriate Factor of Safety numbers (differentiated as FG = factor of guarantee, FS = factor of safety, and FI = factor of insurance) depends very much on cause-effect sequences, and prefailure deformation behaviors, and corresponding "Satisfaction Indices" (Ref. 19).

The same general principles are (should be) valid for soil foundations, and in TRSLS soil one benefits from two facts: (a) generally stress-strain curves are relatively ductile in compression; (b) sampling and laboratory testing always causes unreal brittleness and loss of strength (moderate), whereby, based on laboratory testing we tend to deal with a histogram of strengths truncated at upper values, and thus the favorable FG condition.

In the paper mentioned it was emphasized that with modern heavy hauling and compacting equipment, the trafficability criteria (CBR, bearing capacity, etc...) generally control within the rising fill, in which the rough inspection indices Percent Compaction $96 \lessapprox PC \lessapprox 105\%$ and SPT $\approx 15 - 18$ have proved satisfactory: the importance of saturation $S \lessapprox 95\%$ has been emphasized in comparison with mere indications of compaction water contents as compared with the Standard Proctor optima.

For the sake of brevity these General Reporters would submit a simple comment that essentially the same concepts hold true for the foundation as for the bottom layers of fill; and that in TRSLS soil the result is better than imagined. Time, cementations, make. TRSLS soils of PC $\approx$ 92-95% exhibit strengths equivalent to the same soils as compacted to about 3-4% higher PC values, or foundation SPT values of about 10-13 might function as equivalent to similar compacted fill of SPT values 4 to 5 points higher.

These are only rough preliminary indications to be adjusted by experienced judgement: for the developing tropical world in the 1970's and 1980's they possibly constitute very much better indices than were available in the 1910's to 1940's when the equivalent multitudes of dams proded the development of the Northern hemisphere's counterpart. We do not condone mere use of such crude feasibility-level indices, and obviously insist on appropriate advanced geotechnical research to justify and improve them. But we cannot condone suppressing such reality either.

Comments and discussions are invited.

4.4. Downstream slope stability, full reservoir.

Leme (Ref. 04) emphasizes the case of the Promissão Dam foundation in which a somewhat more metastable "brittle, porous" structure caused the often encountered difference between hypothetical Mohr--Coulomb strength envelopes, at (σ_1'/σ_3')max. vs. at $(\sigma_1 - \sigma_3)$max. failure criteria. One must first assume that in such brittle behaviors the routinely measured pore pressures (at top and bottom of specimens) and routinely computed average stresses and strains, really apply at the failure plane and "instant": we accept the assumption, since the proposed theoretical dilemma would be raised in very many other soils (e.g. Scandinavian and Canadian sensitive clays, and a multitude of other less singular soils, e.g. Magnan, Ref. 12).

It appears to these General Reporters that the decision to accept as general as possible a failure criterion, however nominal (presently the principal stress ratio criterion, much preferred) is a decision of engineering efficiency, besides being as closely as possible one of technological prevailing truth. Thus, it is not to be rigidly obeyed as dogma: for instance, most recent efforts at adjusting theories to case histories in the quick clays seem to be confirming a preference for use of a judicious hybrid combination of effective stress and total stress analyses.

As regards downstream stability, full reservoir, we again refer to the question of "changes of conditions" (de Mello, Ref. 18), and merely jot down the emphatic importance of rate of first filling, first filling stability as compared with long term stability, and the absolute need that for both these situations the dam should really have a Factor of Guarantee FG $\geq$ 1.5 against failure (which should be physically inconceivable). The triaxial shear tests to which Leme (Ref. 04) refers were fully saturated first (by back--pressure) and then sheared at constant volume, fully undrained, all the way from an isotropic condition: in natural conditions there is a time of saturation, and the change of stress obliquities does not start from an isotropic state and may well imply a stress release leading to OCR conditions, and, finally, what matters for excess pore pressures due to incremental shear are the shear strains (not necessarily instantaneous with stress changes).

In short, one may again repeat that the type of problem mentioned affects many a soil and all dams. In the specific case of TRSLS soils there is a favorable tendency not to reach full saturation (the wetting front advances much more rapidly through some macro-pores, and traps air in smaller pores that were bypassed), and, also, the collapsive tendencies guarantee that there is very little swelling (and respective loss of strength) if any, while the dominant tendencies with time are most often favorable (secondary compressions and, unless reservoir waters are chemically unfavorable, some microcementations).

4.5. Upstream slope stability, rapid drawdown.

For the indispensable brevity the Authors limit themselves to referring to the discussions of this special case as presented in the same paper, which maintains the same principle of analysis of "changes of conditions". The Authors know of no specific factors whereby TRSLS soils might be subject to less favorable conditions than other foundation soils. Discussions are invited.

5. FOUNDATION DEFORMATIONS.

5.1. Heave on excavation.

To the knowledge of the Authors there is no mention of perceptible heave of TRSLS foundations on excavation above the water table. Oedometer test data confirm the very salient hysteresis, whereby the unloading curve practically does not swell. Incidentally, a few check tests have revealed that the maximum past pressure is very markedly identified in the routine log plot (and Casagrande construction) of the oedometer compression.

On the other hand, any excavation of saprolites below water table is terribly difficult and damaging to the soil, essentially as in many silts, but even worse. Groundwater lowering proves very frequently inefficient, even with vacuum, principally because of the preferential flow paths typical of saprolites. The soils are very susceptible to instability ("bull's liver" condition) by upward seepage gradients. The use of compressed air stabilizes very efficiently, remarkably: as is currently said, TRSLS soils behave much better in compression and under compressive seepage gradients, than in free water or especially in conditions of seepage gradients generating tension.

5.2. Compressibility settlements.

TRSLS soils have behaved as preconsolidated soils, with some nominal preconsolidation pressure, and a nominal virgin compression index straight line well set out in oedometer tests. Apparently the predicted and observed settlements as computed through such routine tests have tallied reasonably in the very mature, porous horizons, relatively homegeneous. In the dense saprolites the computed settlements have generally been considerably higher than the measured ones, which is what prompted the postulation of the principle of equivalent strains (de Mello, Ref. 20) for attributing greater proportion of participation to the more rigid soil elements.

Since such compressibility settlements tend to be essentially instantaneous, the increments of settlements with time, after topping off the dam, have been quite insignificant (of the order of 10-15% of the settlement due to embankment loading).

Compressions seem to correlate much better with dry density, porosity, or nominal PC%, than with plasticity indices of the pseudo-clays. TRSLS soils behave as granular, even when plastifiable to relatively clayey indices.

5.3. Lateral displacements.

There seem to be no indications of perceptible lateral displacements of TRSLS horizons due to embankment loading. Such a trend appears compatible with the collapsive "granular" behavior of the materials in situ. However, as absence of evidence is not evidence of absence, the Authors invite data and comments.

5.4. Collapse settlements.

One of the topics most often repeated is that of the collapse settlements of porous unsaturated soils (e.g. Pruska, 05, Rabinovich and Korotkova, 28, etc.). The phenomenon occurs in several other materials, predominantly in "dry" conditions conducive to evaporite microcementations between particles and suction, (e.g. loesses, etc..). All such cases are of great concern to earth dams because the differential settlements can be large, are rapid (almost "instantaneous") and occur on first filling, at a most inappropriate time for any tensile cracking to develop transversally.

One added point occurs in a seriously aggravated condition in many TRSLS soils. Since tropical climates are frequently associated with heavy rainfalls and significant infiltrations, the fact is that mere soaking and infiltration seepage gradient is not sufficient to cause the collapse (as is the case with many loesses etc... where foundations are reasonably pretreated by ponding). TRSLS soils collapse under joint effects of loading plus soaking, since they have naturally adjusted to soaking under overburden stress plus downward gradient; and, depending on the nature of the structural bonds and their breaking point, the net phenomenon reaches conditions that are greatly varying in intensity. For instance, in the old-fashioned double-oedometer tests, there is generally a maximized combination of intermediate pressure plus submergence at which collapse is greatest: at low pressures there is no collapse due to soaking, and at relatively high pressures, after the soil has been densified by the immediate compression by load, the incremental compression due to soaking again tends to be small.

One must alert all geotechnicians against relying upon SPT indices to reflect degree of collapsivity: of course low SPT values lead to predictable difficulties, but the trouble is that the opposite is not true. There are TRSLS soils of quite high SPT values that suffer collapse in significant magnitude: the pressure of the breaking point may be higher.

In the Authors' experience the oedometric compressibilities due to collapse tend to be greater than reality: probably because the lateral confinement presumed to be provided by the rigid metal ring is not effective until some minimum compressions force the sample against the wall. It has been found preferable to use triaxial tests, with really controlled lateral stresses, in lieu of pseudo-controlled lateral strains.

One other very important point is that in some cases plate load tests have been used to assess collapsive parameters. In the case of plate load tests, depending on the stress levels there is a dominant condition controlled by bearing capacity failure which has nothing to do with mere collapsivity, being much worse. One need only realize that under an embankment loading there is a linear variation of loading and shear stressing along the foundation plane, while in the plate load tests there is the abrupt discontinuity between σ applied by the plate and zero at the soil surface beyond the loaded periphery, to comprehend the error in use of plate load tests for quantifying collapsivity.

Collapsivity of TRSLS soils constitutes a real challenge, both for index detection, for test quantification, for decision on tolerable limits, and, principally, for the development of creative corrective treatments. The General Reporters have some ideas that would seem worthwhile discussing.

6. FOUNDATION PERMEABILITY AND PROBLEMS.

6.1. Permeability and consequences.

It is general experience that TRSLS foundations are significantly more pervious than the compacted embankments built out of them. Such higher permeability is affected by all three components: mass permeability of the soil, preferential planes of seepage (principally in saprolites), and canaliculi (due to laterization processes, chemical, entymological, botanical, biological). As was emphasized in the Senior Author's Rankine Lecture (Ref. 19), by Design Principle 1, DP-1, the priority treatment has to be of physical exclusion of extreme situations, and therefore of sealing or intercepting the wider open paths: this concept, although embodying a priority design decision, will be discussed under item 8. For the present, discussion concentrates on applying Design Principle 3, DP-3 "...to entice integration of favorable effects within the statistical universe of the foundation horizon". If there is any pervious horizon under the embankment dam, once the validity of behavioral averages (flownet, etc..) has been assured, two solutions have been automatically considered: on the one hand to create a cut off (a "vertical impervious" barrier across the pervious horizon); on the other hand,to increase the seepage path by employing the classical external impervious blanket.

The impervious cutoff element which will be employed whenever practical in no way affects the adoption of the internal impervious blanket, which at worst becomes superbundant and therefore could be relaxed in quality (de Mello, Ref. 18,15). TRSLS soils decrease in mass permeability very significantly by compression: apparently and justifiably more so than in sediments because macropores of TRSLS soils should be the first ones to suffer compression. The internal impervious blanket is strongly recommended, whereas an external blanket suffers from many problems: the internal blanket profits of the favourable permeability gradients generated by compressions, and it employs the pretest and preload principle, since the foundation is more heavily loaded during construction (by the embankment), and when the uplift sets in due to reservoir filling, it causes a decrease of effective stresses. On the other hand, besides the many problems it faces, the external blanket proves to be a conceptually wrong solution, since the critical loading condition for it is during first reservoir filling.

Problems due to foundation permeabilities are uplift pressures dangerous to downstream, higher seepage losses than tolerated or desired, and risks of unfavourable exit gradients leading to piping and erosions. Average treatments, such as the blanket, improve conditions in general, but do not guarantee against the really unfavourable local seeps and their effects on safety. TRSLS soils can be either highly resistant to erosions if cemented (ferruginous, or by insect secretions, etc..) or can also be locally quite susceptible to erosion (volumes chemically lixiviated etc..). The main point is that the range of variations is extreme, all the way from macro-canaliculi (4" in diameter), sometimes erosion-protected by ferruginous coatings but sometimes not, to moderately impervious silty masses susceptible to unstabilization by unfavourable seepage gradients.

As discussed below one must resort to types of grouting treatments for the discontinuities. Constant width impervious diaphragm erosion-resistant walls have to be carefully examined with regard to the concentration of very high seepage gradients around the bottom, therefore requiring erosion-resistant rocky conditions of embedment.

The main recommendation seems to be the control of rate of rise of reservoir on first filling, so that the foundation seeps can easily identify themselves and be promptly treated. The General Reporters invite discussion on this important question.

6.2. Chemical and biological attack with time.

Reservoirs in densely forested areas, and reservoir waters containing different dissolved chemicals might adversely affect TRSLS foundations of dams in long-term conditions. Little is known to have been observed or published on the question. In the same light one should be on guard against continuing biological activities during periods of reservoir lowering.

6.3. Saturation and liquefaction under seismicity.

The question does not seem to have been approached. It would seem, however that TRSLS soil horizons might prove reasonably protected from such problems both because of wide variation of pore diameters, and relative difficulty of saturation to the critical degree, and because of beneficial effects of cementations.

The subject invites serious discussion.

7. DESIGN AND CONSTRUCTION SPECIFICATIONS FOR CORE CONTACT ON TRSLS SOILS.

A problem that has been the source of very acrid debates and litigations is the specification on depth of excavation and quality of horizon for core contact.

As is well known, the principal factors affecting acceptance of core foundations definitely do not include shear strength nor hypothetical upstream-downstream shear displacements. Therefore one might reason that the core foundation could rest on a dense saprolite or weathered rock, <u>at a higher elevation</u> than the adjacent shells. The principal factors, however, that have determined the almost general requirement of excavating core contacts to definitely greater depths and sounder rock than the adjacent shells, may be interpreted as being three.

Firstly the desire to reach "sound groutable rock" for a guarantee against any risk of "piping", that is, by guaranteeing that if the rock has any wider open cracks, they will be adequately grouted, avoiding the piping of any core material through them. Secondly, to achieve a carefully sealed contact at the core-rock transition, partly to reinforce the above behavior, partly to avoid any dangerous preferential seepage path along the contact. Thirdly, to achieve, for the core, conditions of settlements that are (a) smaller than those of the adjoining transitions and shells, so that there will be no core-hangup (b) as nearly as possible longitudinally uniform or gradually varying, so as to avoid differential settlement tensile cracking. Let us simplify by calling them the (1) criterion of groutability against potential piping, (2) base permeability criterion, subdivided into permeability across the core, and preferential path contact-permeability and potential pipe-ability (3) settlement criterion. (Fig. 7).

The contact-quality criterion will be discussed here, since the other two merit special separate consideration. If we accept for a moment that modern specialized grouting practices are proving adequate for thorough grouting of weathered rocks and saprolites, both in open cracks and in potential discontinuities easily opened by hydraulic fracturing, then the classical dictum ("sound groutable rock") should well be limited to "firm groutable in situ

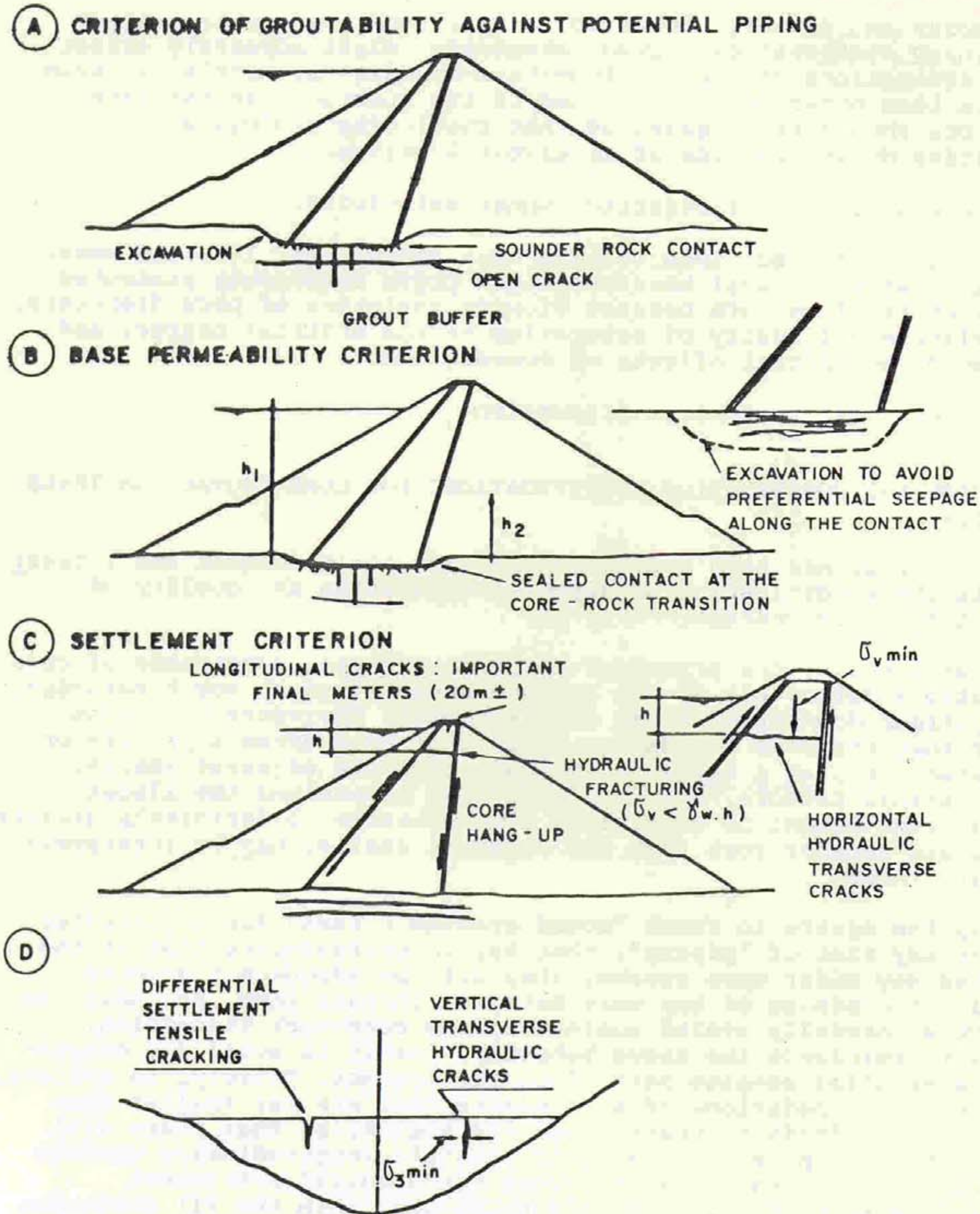


FIG. 7 SCHEMATIC SUMMARY OF PROBLEMS POSSIBLY GENERATED BECAUSE OF FOUNDATIONS UNDER CONSIDERATION (APUD FIG. 3, REF. 15).

material" in a manner to achieve a good impervious adherence between all horizons in the projection of the core and into its foundation.

Well, one of the most undeniable experiences in saprolite foundations is that the decision to support the core on a firm foundation excavated mechanically to definite geometric grade is not only acceptable, but far preferable, technically and economically, to the attempt to excavate down to a "sound rock surface" (cf. Fig.8). It is characteristic and inevitable that the sound rock surface will be found in a most irregular topography (including reentrances and negative slopes etc..): this causes great difficulties, technical and logistic (therefore economic) to achieve the necessary perfection of cleanup (air-jet) and compacted backfill. Note that very frequently it has been preferred to use generous applications of dental concrete and even a thin layer of minimum concrete filling over the entire surface, deeper points receiving greater thicknesses.

Technically, after a fairly continuous sound rock surface has been encountered, there is still quite a probability that underlying zones of differentially weathered rock will persist, and thus the effort to reach sound rock (presumed necessary for groutability) will be futile, because underlying weathered seams will still require the presumably suspect grouting. Moreover, technically, no compacted fill can be as thoroughly transitioning into the sound rock as a continuum, as the weathered material transitions into its own parent rock. The attempts to apply coats and transition layers between the foundation and the core always present serious doubts regarding how such two additional discontinuities will behave under load, water pressures, and time. Specifically for instance the use of dental concrete for rock cavities is unquestionable inasfar as the concrete achieves an extension of rock-like behavior up to the desired grade or smoother support surface: however, if the concreted or gunited thickness is applied in an overall treatment of highly varying thickness and rigidity, it should be visualized as subject to cracking, if there are, under the sound rock surface, the differentially weathered (differentially compressible) bands of rock.

In short, the overall problem hinges inexorably on developing acceptable grouting both of saprolites and weathered rocks, whenever necessary, and of the cracked sound rocks.

In the investigation and design phase, in many a saprolite (granitic, etc..) the apparent need arises to establish the foundation elevation based on the percent core recovery. As is well known, each corestone is absolutely sound, giving 100% recovery of the "chord" traversed by the boring: on the other hand the material in between successive corestones is generally a saprolite.

Questions that matter considerably to the Contractor, and to the Field Inspection (attempting to follow designer's specifications), are: (a) the estimation of the total volumes (and probable dimensions) of corestones within the volume to be excavated; and (b) the estimation of the probability of a certain percentage of core contact area (undulated, elephant-back corestone exposures) along a given (roughly) geometric plane (excavation plane, pre-established for Contractor's needed planning). The principles at play are of quantitative stereology (e.g. Ref. 32). The borings establish some

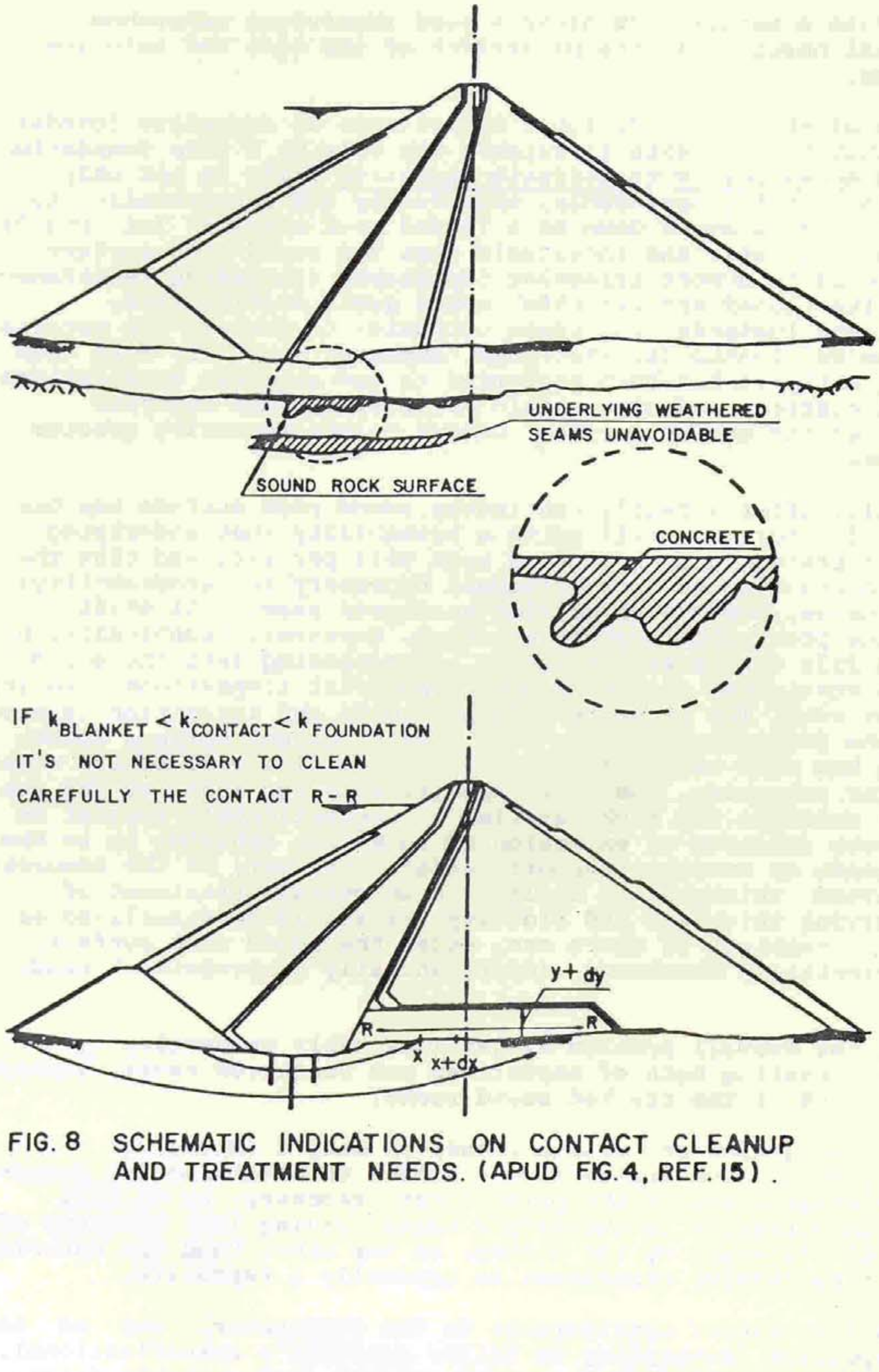


FIG. 8 SCHEMATIC INDICATIONS ON CONTACT CLEANUP AND TREATMENT NEEDS. (APUD FIG.4, REF.15).

indications of occurrences of (vertical) LINEAL FRACTION dimensions: from such data, assuming (uniform) spheres, what are the statistical probabilities of the Lineal Fractions themselves (borings are always few and distant), and of VOLUME FRACTIONS (in the excavated volume, and below the foundation plane), and of the AREAL FRACTION along the foundation plane, for the α-phase (corestones) in comparison with the β-phase (matrix)?.

The problem in TRSLS soils (saprolites is also technically important because the horizon of transition between saprolite and the cracked partly weathered bedrock generally behaves as the dominant pervious horizon (e.g. Ref. 23): and this horizon is the most difficult one to grout satisfactorily, until very recently being considered impossible to grout.

The problem of quantitative stereology has been analysed occasionally for almost a century (Rosiwal, German geologist, 1898) and by some of the greatest names recognized in our profession (Ref. 08): moreover, it is of great interest to many another field (e.g. metallography etc...). The two extreme solutions, of fully statistical mathematical probability of very great numbers of data and assumed normal distribution, and of very few (e.g. one) α-particles in the β-matrix, lead to the widely different conclusions:-

(1) numerical equivalence of L, A, V proportions

(2) proportions of L, A, V as functions of L, L^2, L^3.

The subject merits considerable additional study and discussion, since for the practical cases of dam foundation investigation our data lie much closer to the scant case (2) than to the statistical case (1).

8. SPECIALIZED GROUTING IN TRSLS HORIZONS.

The experience with specialized grouting of saprolites and laterites arose recently with the desire to treat erratic holes (canaliculi, of mm to about 10cm diameters) that abound in some laterite horizons to great depths (20-25m) in a manner as yet rather indeterminate, unpredictable, and unexplained. One hypothesis is that their origin is associated with ancient termite colonies. Whatever the hypothesis, it is obvious that the probabilities of some holes (canaliculi) being found by other holes (borings or groutholes) are most remote: the principle of successful grouting of cracked rocks is that it is reasonable to anticipate groutholes crossing planar cracks (joints etc.) and thereupon the grout travel (liquid) ensures taking up the open planes crossed. In a reasoning of direct antithesis it was planned to create planes of hydraulic fracturing by the grouting: thus, not only would be horizon the generally benefited in imperviousness by the grouted planes, but also the canaliculi that would be perchance crossed by the grouting planes, would also have adequate opportunity of serving for some grout travel and consequent sealing. The improvement of the unsaturated soil mass (any saprolite irrespective of lateritic canaliculi) crossed by grouted planes of hydraulic fracturing is twofold: partly there is a compression (consolidation grouting) of the soil volume by thickening hydraulic fracture widths; and partly, each grouted plane would itself act as a relatively impervious sheet, all directions of planes within the treated zone (hopefully somewhat crisscrossing) being favourable except those that would probabilistically lie vertical in directly upstream-downstream.

It must be emphasized that the problem of such tubular and micro-cavernous discontinuities in TRSLS horizons under dams, and very long dykes, has been proving to be most challenging and often frustrating at all phases schematically visualized: Detection; Investigation; Problems; Solution Options; Decision; Minitoring Planning; Anticipated Corrective Action. In one case where water losses are feared to be the principal problem, whereas moderately slow water rise in upper few meters is anticipated as a principal element of safety, a long enough blanket for the extensive dykes is expected to eliminate 90-95% of preferential flows, and final activity is postponed to accompany the few months of final reservoir filling. Economics of energy production has considerable influence. In another case, where planar hydraulic cracking is more assured (saprolites) and more intensive foundation treatment seems justified and necessary, the special grouting technique has been employed.

In planning the selective grouting it was obvious that one must resort to the "tube-a-manchete" technique, at least for an adequate experimental grouting program. The principal technical problems were associated with (1) selection of clay-cement mix for the execution of the grouthole sheath, to be cracked at each sleeve; (2) the selection of adequate pressures for causing a hydraulic fracturing sufficient for grout travel across the distance between adjacent holes, but limited so as not to propagate the cracking too far; (3) choice of a judicious criterion for bringing the grouting of a given hole and manchete to a stop, and choice of compatible criteria for the possible introduction of complementary grouting in the same hole and/or in complementary holes in zones appearing to be too pervious; (4) selection of appropriate clay-cement mix for the grouting of the cracks. For both the clay-cement mixes (1) and (4), the desired mix is such that after an initial set as rapid as possible, the strength and deformability of the grouted sheath and sheets, should not be noticeably differentiated from the surrounding ground: strengths and rigidities greater than the surrounding soil horizon would invite unfavourable stress and strain redistributions, and subsequent cracking, when the horizon compresses under the embankment loading.

It is beyond the intentions of this General Report to delve into the developmental details that gradually achieved a treatment apparently meeting the foundation design requirements, as proven by careful inspection of the faces of several trenches, both during grouting and grout emergence, and after adequate set of the several grouts. Differently coloured clay-cement mixes were used, both to differentiate grouts forced out of different manchetes, and to differentiate grouts forced out of the same manchete in distinct stages. Some of the interesting data collect- ed from the extensively observed trials are somewhat documented in the tables, graphs and photographs. In short, selective grouting by hydraulic cracking emanating from manchetes in the tube-a-manchete technique has been successfully achieved. In some horizons (principally the upper residual soil) the cracked planes were most frequently vertical, but in the saprolites, frequent cracked planes were inclined and parallel, suggesting σ_3 planes associated with relict joints. One curious observation was that quite repeatedly successive phases of grouting developed planes very near to previously grouted ones, even when the grouted thicknesses reached as much as 1-2 cms: it would thus seem that despite significant compressions, the preferential planes of minimum σ_3 may not change. In the case of a continuous grouting with successive coloured grouts, it was found that the tongue of new grout wedges in the center of the previous grout, pushing it outwards. Many canaliculi were encountered well grouted, as desired; in some cases where apparently the grouting did not extend long enough to fill canaliculi completely, it was found that the partial grouting formed a tubular infilling, with central hole. The criteria for grout volumes and pressures intended to limit travel of cracking and grouting, still need judicious adjustments: it has been found for the present that at the pressures and rates of pumping used, the grout pushes forward essentially with no loss of head, and therefore stoppage has to be provoked either by an arbitrary limitation of grouted volume, or by significant reduction of pressure. Clay-cement mixes have been achieved that are remarkably compatible with the surrounding soil, as judged by meticulous tactile inspection cutting across the inspection pit faces with a pocket-knife.

The important point is that one does not need to visualize the impervious treatment of such saprolites and laterites as including, as only alternates, either the excavation and recompaction of very deep cutoff trenches, or the execution of deep clay-cement slurry-trench cutoff walls extended by cement grouting of the underlying rock, or the use of the yet relatively expensive chemical grouting possibilities. The proof of the success of the treatment will only become available after reservoir filling. The General Reporters submit this very crucial novel problem to comments and debate.

9. FINAL COMMENTS

The purpose of a Conference, and of such a General Report, is not to set down the presumed dogmas applicable, but to raise questions recognized as open, or as requiring honest reopening. One main ecological question that persists, beyond the details above mentioned, is to what extent will future entymological, animal, botanical, bacterial and chemical activities change, increase, interfere with the designed and constructed conditions, controlled for little more than first filling. Nature will have to teach us, and hopefully many collateral professions have much to contribute. If it is true that civil works are live and dynamic, we cannot fail to emphasize the recognition that Tropical regions are recognized to favour life's dynamics in a much more accelerated pace than in other ecologies. We shall have to monitor consciously, under the wise dictum that an engineer must always be prepared for surprise, but hopefully not for dismay.

The detailed questions submitted for discussion have merited being summarized. We trust that the oral and written discussions will have much to contribute.

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REFERENCES

(01) CADMAN, J.D. and Buosi, M.A. "Tubular cavities in the residual lateritic soil foundations of the Tucuruí; Balbina and Samuel Hydroelectric dams in the Brazilian Amazon region". 1st International Conference on Geomechanics in Tropical Lateritic and Saprolitic Soils, Brasília, Feb. 1985, v. II, p.111.

(02) CARRIER, III D.W., et al. "Optimization of mine-waste disposal and reclamation in the Amazon region". loc.cit, v. II, p.123.

(03) DIB, P.S. "Compressibility characteristics of tropical soils making up the foundation of the Tucuruí Dam in Amazonas (Brazil)". loc.cit, v. II, p.131.

(04) LEME, C.R. de M., Chapter 3.4 "Dam Foundation". Draft, Special Report of the ISSMFE Committee on Tropical Laterite and Saprolite Soils, 1985.

(05) PRUSZA, Z.V. "Miscellaneous aspects of Guri soils as foundation materials", loc.cit, v. II, p.165.

(06) DOS SANTOS, O.G. "Experimental grouting of residual soil of the Balbina earth dam foundation - Amazon, Brazil". loc.cit, v. II, p.143.

(07) DE SOLA, O. "Weathering in the Guri area - Venezuela". loc.cit, v. II, p.155.

(08) BISHOP, A.W. "The influence of an undrained change in stress on the pore pressure in porous media of low compressibility". Geotechnique, v. XXIII(3), p.435, 1973.

(09) EIDE, O., et al. "Unconventional concepts for dykes and dams on soft clay foundation". Canadian Geotechnical Journal, v. XXI(3), p.581, 1984.

(10) GARGA, V.K. and Aboim Costa, C. "Stress compressibility characteristics of a residual soil from gneiss". IX ICSMFE, Tokyo, 1977, v. I, p.105.

(11) JURGENSON, L. "The application of theories of elasticity and

plasticity to foundation problems". 1934, Contributions to Soil Mechanics, BSCE, 1925-1940, p.184.

(12) MAGNAN, J.P. et al. "Étude en laboratoire des états limites d'une argile molle organique". Revue Française de Géotèchnique, nº 20, p.13, 1982.

(13) DE MELLO, V.F.B. "Site Investigation and Foundation Decisions for Offshore Structures", Opening Address. 8th South-East Asian Conf. SMFE, Kuala Lumpur, 1985.

(14) DE MELLO, V.F.B. "First-order specifications for support of embankments an tropical saprolites and laterites (in print), 1985.

(15) DE MELLO, V.F.B. "Problems and solutions for embankment dam foundations on weathered soil horizons and cracked rock". IIIrd Geotechnical Seminar of the Colombian Society SMFE. Bogotá, Aug. 1984.

(16) DE MELLO, V.F.B. "Desafios no desenvolvimento de uma engenharia de solos autóctone firmemente enquadrada em princípios universais". Keynote Lecture, 7th Brazilian Conf. SMFE, Recife, 1982, v. VIII, p.47.

(17) DE MELLO, V.F.B. "Comparative behaviors of similar compacted earthrock dams in basalt geology in Brazil". Int. Symposium on problems and practice of dam eng'g in Asia, Bangkok, Dec. 1980, p.166.

(18) DE MELLO, V.F.B. "Some problems and revisions regarding slope stability assessment in embankment dams". Int. Symposium on problems and practice of dam eng'g in Asia, Bangkok, Dec. 1980, p.81.

(19) DE MELLO, V.F.B. "Reflections on design decisions of practical significance to embankment dams". 17th Rankine Lecture, Geotechnique, 27(3), p.270, Sep. 1977.

(20) DE MELLO, V.F.B. "Thoughts on soil engineering applicable to residual soils". 3rd South-East Asian Conf. on Soil Mechanics, Hong Kong, 1972, p.5.

(21) DE MELLO, V.F.B. "The Standard Penetration Test". State-of--the-art report. 4th Panam. Conf. SMFE, Puerto Rico, 1971, v. I, p.1.

(22) DE MELLO, V.F.B. et al. "True representation of the quality of a compacted embankment". First Panam. Conf. SMFE, México, 1959, v. II, p.657.

(23) NAKAO, H. et al. "Percolações preferenciais nas fundações das barragens sobre maciços basálticos". Simpósio sobre a Geotecnia da Bacia do Alto Paraná, São Paulo, 1983, v. IIB, p.425.

(24) NAKAO, H. et al. "Recalques de fundações de barragens sobre maciços basálticos". Simpósio sobre a Geotecnia da Bacia do Alto Paraná, São Paulo, 1983, v. IIA, p.233.

(25) OLIVEIRA, H.G. et al. "Desempenho das fundações e maciços de barragem de terra e enrocamento de Itumbiara". VII Congresso ABMS, Recife, 1982, v. VI, p.101.

(26) POULOS, H.G. and DAVIS, E.H. "Elastic solutions for soil and rock mechanics", J. Wiley & Sons, Inc., 1974.

(27) QUEIROZ, L.A. "Compressible foundation at Três Marias earth dams", First Panam. Conf. SMFE, México, 1959, v. II, p.763.

(28) RABINOVICH, I.G. and Korotkova, O.N. "Calculation of the local deformations in collapsible soils". VII Conf. Polish Soc. SMFE, Poznan, v. I, p.345. (1984).

(29) SCHERRER, H.U. "Dam foundation settlements due to saturation". Bulletin 2, ABMS, p.15, May 1965.

(30) SILVESTRI, V. "The bearing capacity of dykes and fills founded on soft of limited thickness". Canadian Geotechnical Journal, v. XX(3), p.428, Aug. 1983.

(31) TOWNSEND, F.C. and Gilbert, P.A. "Residual strength test, Paraitinga and Paraibuna Dams, Brazil". Misc. paper - S-74-21, US Army WES, Vicksburg, Jun. 1974.

(32) UNDERWOOD, E.E. "Quantitative Stereology", Addison-Wesley Publishing Co., 1970.

(33) VAUGHAN, P.R. and Kwan, C.W. "Weathering structure and in situ stress in residual soils". Géotechnique, 34(1), p.43, Mar. 1984.