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Comparative behaviors of similar compacted earthrock dams in basalt geology in Brazil

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SYNOPSIS. One important problem in higher compacted earth-rock dams concerns compatibility of settlements between core and rockfill. For such compressibility behavior, adjustment coefficients between laboratory and field are sought. It is shown that both the very clayey cores and the sound angular compacted rockfill exhibit nominal compaction precompression followed by virgin compressibility. The magnitudes of settlements in both materials have proved very similar. Statistical correlations are offered for preconsolidation pressure and compression indices of undisturbed block sample specimens of compacted clays. Construction pore pressures are very low.

1. INTRODUCTION

The question of compatibility of settlement deformations in zoned embankment dams is well recognized as one of the problems that significantly condition satisfactory embankment dam design and behavior. Moreover, there is every theoretical reason to anticipate that such a problem should be amenable to direct and rapid adjustments of the necessary testing and consequent interpreted stress-strain parameters: settlements derive from integrations of compressibilities of ΔH Soil elements, and are therefore definitely associated with statistics of averages. The purpose of the present paper is, however, to further advance my own earlier indications on how significant an adjustment has to be made in deriving the appropriate mental model and geotechnical design-prediction parameters because of historic misconceptions on the significance of compaction, and because of premature crystallization of conventional soil mechanics on would-be standardized routine test procedures, interpretations, and design computations.

In fig. 1 I submit once again (cf. Tokyo ICSMFE 1977, Vol. III) a schematic visualization of what I have called the Experience Cycle in Civil Engineering. No matter what degree of crudences of the "data" on which we have to base our design decisions, such decisions really begin by establishing a "physical model" intended to fulfill "equal to or better than" the desired functional behavior. That is why I emphasize that Civil Engineering is based on "prescriptions", themselves based on predictions of what will not happen, rather than restrictively on difficult predictions of what exactly will happen (Rankine Lecture 1977). For instance, on the basis of the earliest identification and Index Tests we already associate certain likely behaviors with the word "clay" and "rockfill": and thereupon from the most preliminary phase of investigation - identification, would not hesitate to "design" a presumed earthcore-rockfill dam section at sites blessed with a red porous clay of presumably high plasticity for a core, and a sound dense basalt from required excavations for compacted rockfill shells.

Such was the condition in 1969 when the Salto Osorio dam was submitted for construction bidding. The Consulting Board had considerable concern about the anticipated high compressibilities of the clay core in comparison with the rockfill shells, and directly decided to change the central core to an inclined-core section. A posteriori finite element analyses and geotechnical tests were engaged (cf. ICOLD, Madrid 1973). Suffice it to mention that the reasonable parameters proposed by experienced specialists were E values of

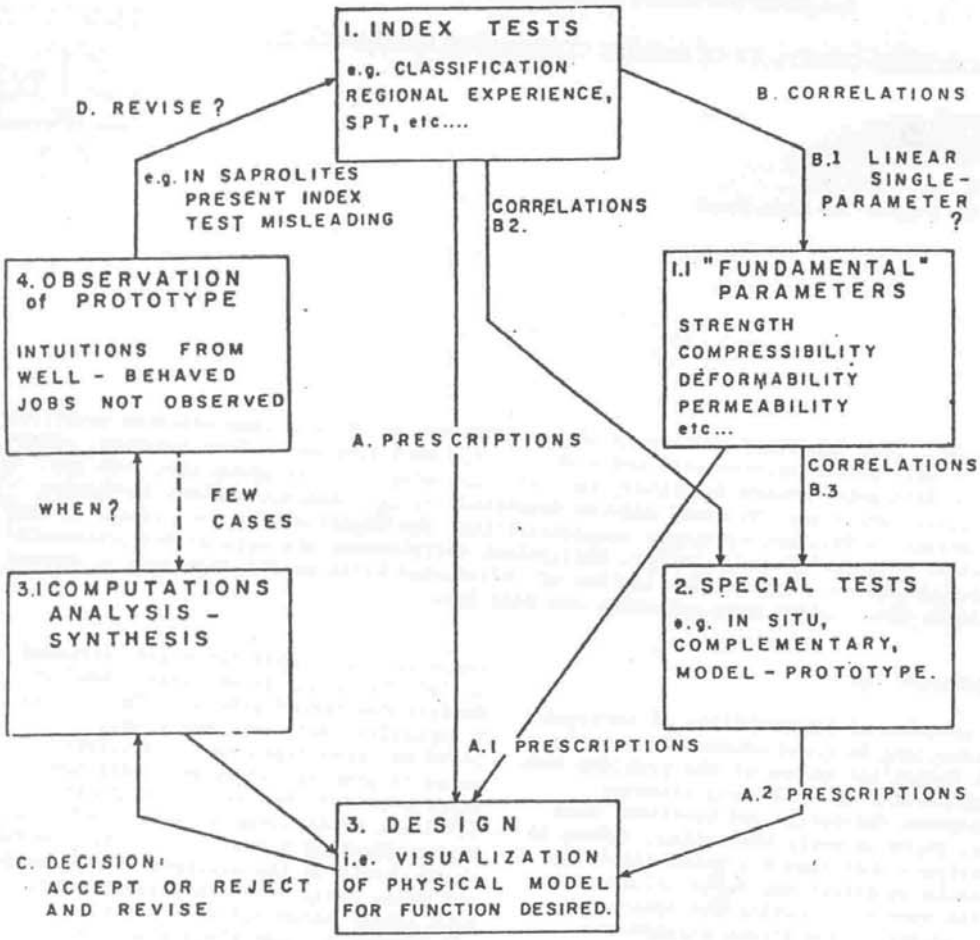


FIG. 1 EXPERIENCE CYCLE IN CIVIL ENGINEERING

about 80 kg/cm^2 for the clay and 1000 kg/cm^2 for the basalt. Careful geotechnical testing was undertaken (cf. steps 1.1 and 2 in Fig. 1) and obviously the case called for careful monitoring of the prototype behavior (step 4). In rapid succession two similar higher dams, Salto Santiago 80m, and Itauba, about 100m at deepest points, followed the same design trend. The data on these three dams are further enhanced, as regards the behavior of the sound dense well compacted basalt, by the monitored data from the Foz do Areia 160m concrete-face dam, in many respects a record-breaking case. Figs. 15 and 16 summarize the design crossections of these four dams used herein to confirm my views on needed revisions on interpretation of behavior of compacted embankment dams.

2. REVISING THE PREVAILING MENTAL MODEL ON THE NATURE OF COMPACTION

It is quite comprehensible that the historical mental model on the nature of compaction should have been, in analogy with the understanding of dense vs. loose sands, a hypothesis of developing through densification a material homogeneously improved. All the publications (predominantly from laboratory research) through the 1950's and 1960's, attributed to each compaction condition (defined by percent compaction PC%, and water content deviation $\Delta w\%$ from optimum) a presumed intrinsic new quality, such as schematically represented in Fig. 2 as regards shear strength. Even meticulous and sophisticated research on "structure of compacted clays" as dependent on compaction parameters (truly second-order effects in much civil engineering construction) always associated all specimens of a given set of compaction indices as pertaining to a common statistical universe. I have repeatedly emphasized that in unsaturated clays and rockfills (materials with significant hysteresis of absorption of compressive energy) compaction really represents the introduction of an apparent preconsolidation effect, with little influence on the C_c behavior of the materials at pressures beyond the preconsolidation pressure, i.e. in the nominal virgin compression range. In Fig. 3a are reproduced a few of the many laboratory tests conducted with regard to different residual clay-silt-sand soils using specimens molded at different (PC%, Δw%) compaction indices: bearing in mind inevitable statistical dispersions the conclusion repeatedly extracted has been that within the narrow range of compaction indices within which a dam is constructed, we may accept that C_c is for practical purposes independent of (PC%, Δw%) for each soil.

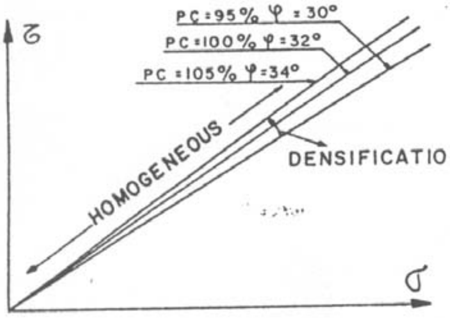


FIG. 2 HISTORICAL MENTAL MODEL OF COMPACTION EFFECT

Returning for a moment to Fig. 1 I wish to emphasize that on looking back at the historical development of soil mechanics (as of any other technology) it is quite comprehensible that both Index Tests (for identification and classification) (de Mello 1979) and also the principal tests for Fundamental Parameters, and even the further Special Tests, should have arisen one by one, without much regard to compatibilities between them. Also, it is inexorable that each test is derived from a preconceived mental model of how the specimen behavior will be incorporated into the integrated prototype behavior. It is inexorable, therefore, that the historical tests should require considerable revision

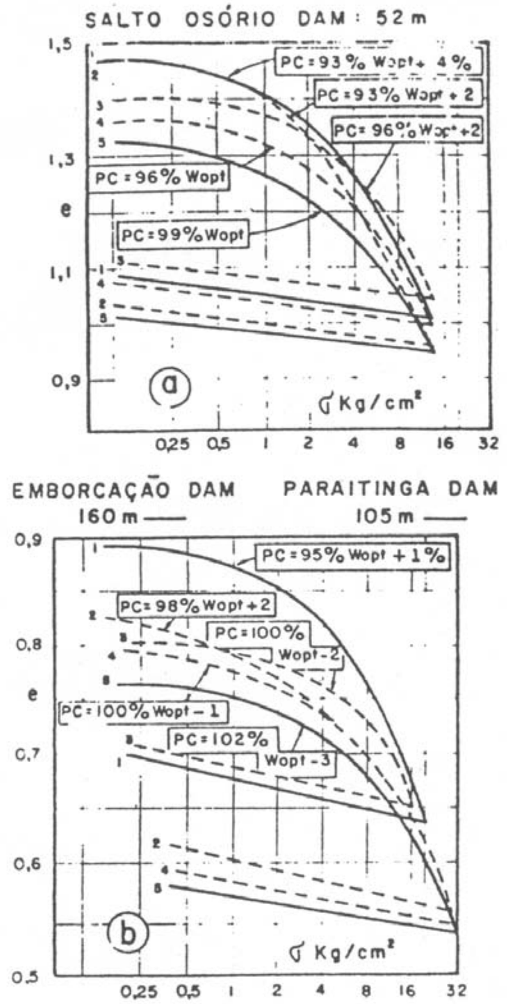


FIG.3 COMPACTION IMPLIES PRECOMPRESSION WITH PRACTICALLY NO EFFECT ON VIRGIN COMPRESSION INDEX.

and readjustment, both for compatibilities between them, and for adequate estimation of the behavior anticipated for new projects. It is firstly sad that in many quarters the need for such recycling revision has neither been promoted nor even perceived. Much more regretable, however, is it when additional indices are lightly suggested without recourse to the context of theoretical and practical knowledge already accumulated: Fig. 4 exemplifies some immediate conceptual questions that surround a recently proposed index, the Dispersion Index, regarding a

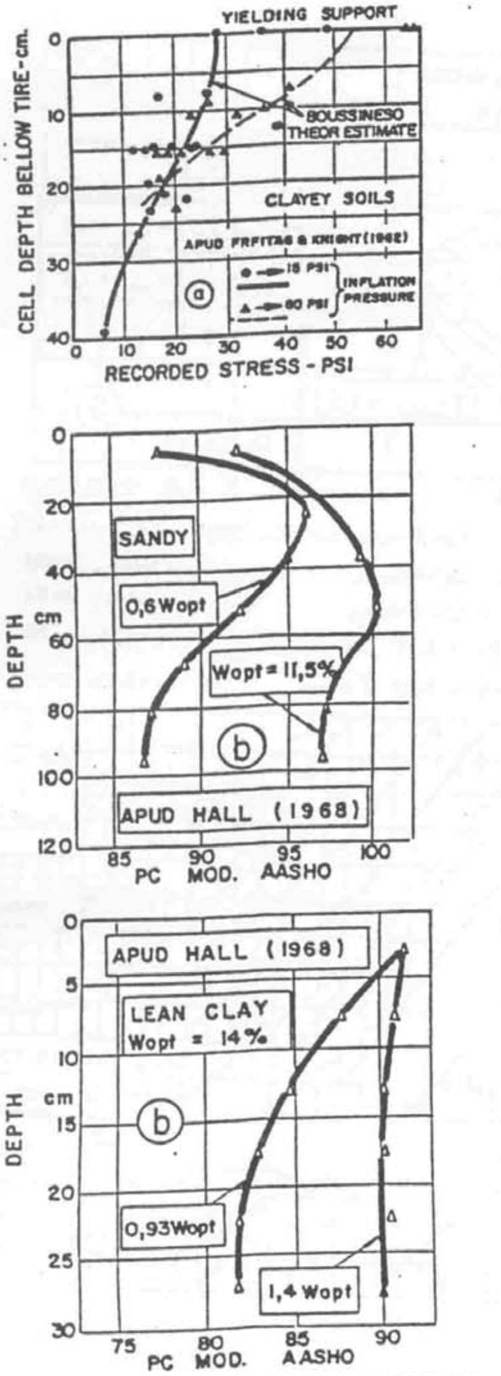


FIG. 6 PRESSURE DISTRIBUTIONS WITHIN LIFT DUE TO PNEUMATIC COMPACTION

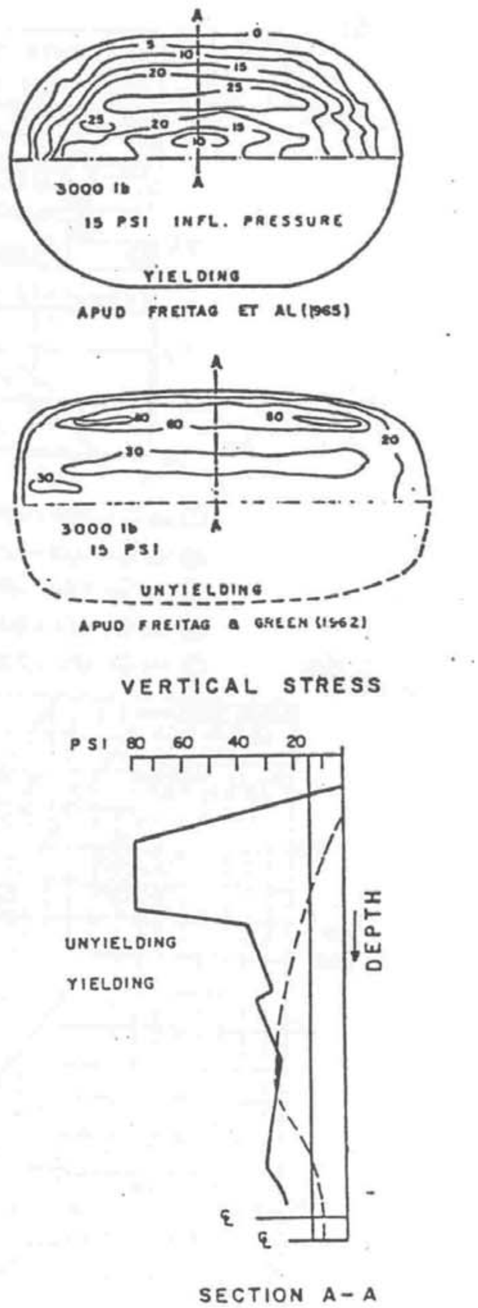


FIG. 7 APPLIED AND TRANSMITTED PRESSURES

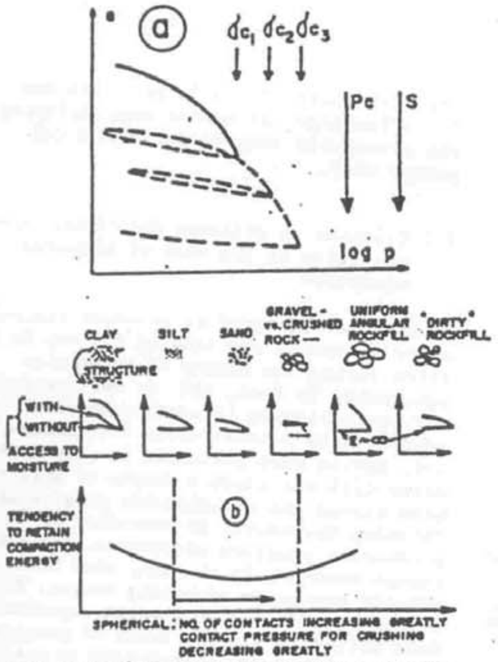


FIG.8 SCHEMATIC COMPARISON OF COMPACTION GRAIN STRUCTURE AND COMPRESSIBILITY HYSTERESIS

ing the specimen in oedometer tests).

Silts, sands and rounded gravels are materials in which there is least absorption of compactive prestress. In the case of coarser granular material the compressions are due to the very significant difference between (point to point) intergranular stresses (as differentiated from the nominal average effective stress obtained by definition and dogma): therefore a crushed aggregate is obviously more compressible than a gravel of same grainsize, and also obviously a clean angular sound rockfill should be more compressible than crushed aggregate of the same rock. Also under the same reasonings it is quite understandable that a "dirty rockfill" should be less compressible than the corresponding clean angular rockfill.

Finally, it should be of considerable interest to note (for finite element analyses etc...) that in the coarse angular materials of compressibility generated by crushing of point contacts, the E value for stress release may be essentially infinite.

Some of the very significant observations in modern compaction of clayey materials are: the fundamental importance of densities and degrees of saturation S% of the clay nuclei in situ (in the borrow pit); this is schematically illustrated in Fig. 10.

Moreover, one faces the problems of bearing capacity both of earthmoving equipment and of compactability of lifts; and the difference between laboratory compaction within metal molds, and the compressive-shearing compaction of a lift in the field (Fig.11c).

Such observations have become more and more salient as equipment greatly increased in capacity and weight, and as one advanced in the use of much more clayey (and/or less unsaturated) borrow materials. Excavations in borrow pits presently often advance through materials much denser than the Proctor maxima, and at S% values absolutely incompatible with any compaction (one only compacts the air voids, principally the macro-airpores): and one always deals with clay nuclei and not with the "totally disintegrated" particle sizes. Ironically the $\Delta w\%$ index would indicate a need for adding water: in a dense, close-to-saturated clay nucleus, the mechanical action would be remoulding and not compaction.

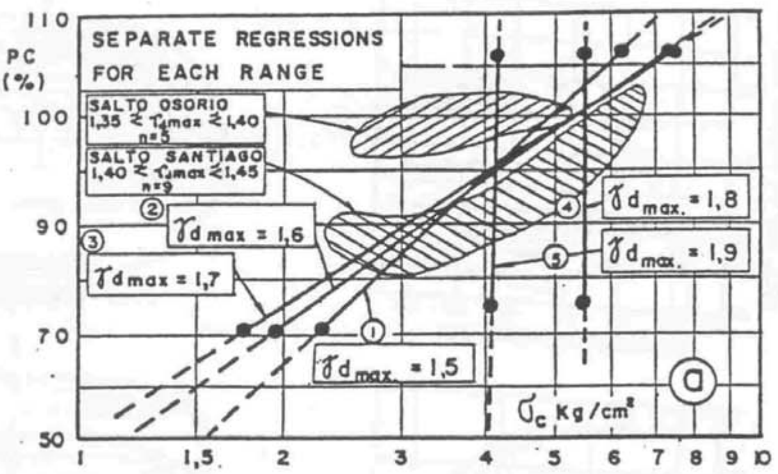
The data from the Salto Osorio, Salto Santiago, Itauba claycore material have been used in Figs. 9, 11a, 11b to demonstrate that the existing theoretical and experimental background can be readily used for such evaluations of compaction and bearing capacity compactability limits. The effective stress envelope from Fig. 9a has been transformed into total stress equations (Fig. 11a) by use of the pore pressure data for two different S% from Fig. 9c. In Fig. 11b we reproduce a set of curves of surface plate bearing capacity σ_{rut} (de Mello, 1969). Finally against such a background we can plot the probable trajectories of compaction of the two different lifts, recognizing that in each lift the S% increases (and ϕ_a decreases) with increasing passes.

2.4 Statistical estimation of virgin compression C_c of compacted clay-silt-sand materials

Having discussed and estimated the compaction pre-consolidation σ_c, we must now establish the C_c values for the purpose of extrapolations of behaviors beyond the precompression benefits of the compaction PC%.

The same oedometer tests on undisturbed block samples were used for statistical regressions that are summarized in Figs. 12 and 13. All points are plotted, and appear to configurate too wide a dispersion: it must be repeated, however, that we are principally interested in the average, and the respective coefficients of correlation are very good. The first obvious attempt

5a.



① $\log \sigma_c = -5{,}35 + 0{,}017\ PC + 2{,}92\ \gamma_{d\,max}$ CORREL. COEF. $r = 0{,}75$, $n = 45$

② $\log \sigma_c = 0{,}25 + 0{,}022\ PC - 1{,}07\ \gamma_{d\,max}$ $r = 0{,}53$, $n = 52$

③ $\log \sigma_c = 4{,}67 + 0{,}024\ PC + 1{,}77\ \gamma_{d\,max}$ $r = 0{,}63$, $n = 61$

④ $\log \sigma_c = 4{,}53 + 0{,}00029\ PC - 2{,}12\ \gamma_{d\,max}$ $r = 0{,}53$, $n = 29$

⑤ $\log \sigma_c = 1{,}50 + 0{,}00031\ PC - 0{,}48\ \gamma_{d\,max}$ $r = 0{,}29$, $n = 50$

5b.

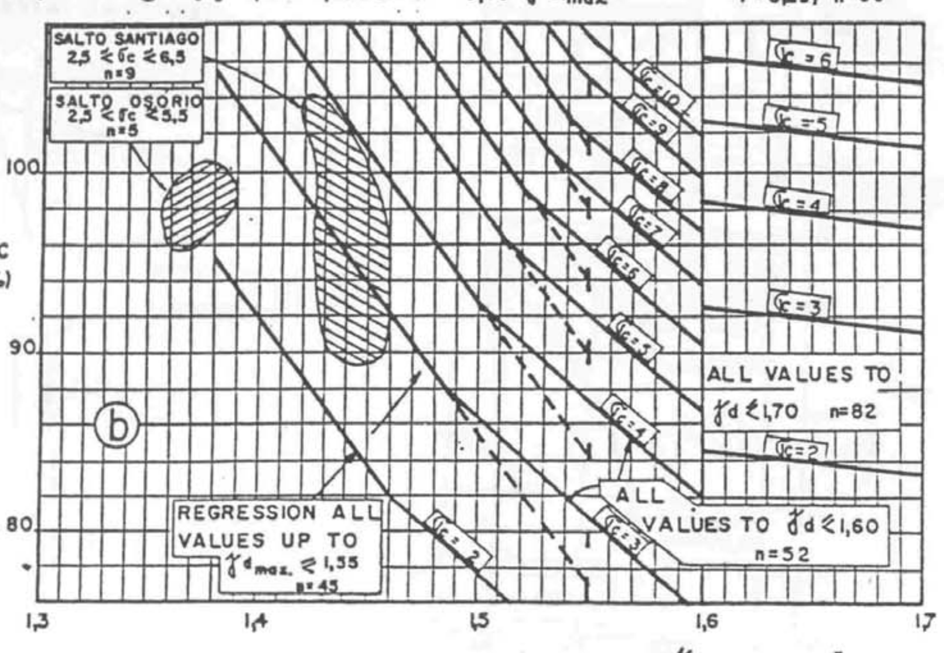
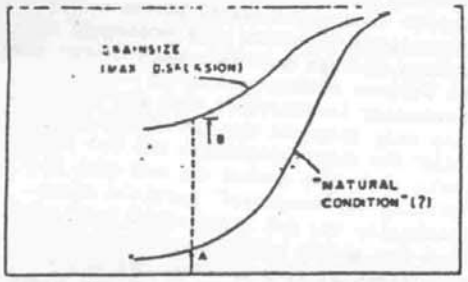


FIG.5 BASES FOR ESTIMATING PRECONSOLIDATION PRESSURES OF COMPACTED SOILS

problem that is in some parts emphasized as afflicting compacted clay dams.



DISPERSION INDEX DEFINED AS A/B

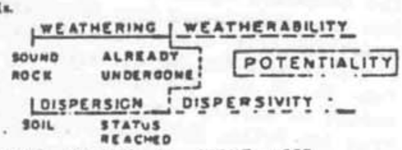
QUESTIONABLE

1 - NOT STANDARDIZED HOW TO TEST "NATURAL CONDITION"

2 - WHAT INTERESTS IS DISPERSIVITY, NOT DISPERSION. DISPERSIVITY = POTENTIALITY FOR DISPERSION, MORE APPROPRIATELY B-A OR B/A (INDEX INCREASES IN DIRECTION OF CORRESPONDENT PROPERTY)

3 - DISTINGUISH TERMINOLOGY

Ex.



4 - DISPERSION DEPENDS ON AGENT USED

FIG 4 EXAMPLE OF CONFUSING PROPOSAL OF INDICES FOR SOIL BEHAVIOR

2.1 Estimates of compaction preconsolidation pressure σ_c from oedometer tests on undisturbed block samples from various dams

The results of oedometer tests on 167 block samples from various Brazilian dams have been analyzed statistically for correlations between PC% and the corresponding preconsolidation pressure. The simple conclusions are summarized in Fig. 5. The correlation depends somewhat on the classification of the soil, as indicated by the maximum Proctor dry density. As can be readily understood the correlation is not good in the sandier materials. The same regressions are furnished in the lower graph in a manner that should make it easier to estimate for any given maximum Proctor dry density and corresponding PC%, what to expect as a compaction preconsolidation pressure.

In the two graphs of Fig. 5 we have plotted the zones corresponding to 14 tests obtained more recently in the compaction of the clay cores of the Salto Osorio and Salto Santiago earth-rock dams employing the presumably very clayey basalt red porous clay.

2.2 Evidence of pressure distributed into the lifts by the tire of compacting equipment

No data can be found on pressure transmitted by sheepfoot and tamping rollers to the lifts during the passes of compaction equipment. In fact, all or most pressure cell measurements in earth dams have preferred to protect cells from damage, and, having been installed in pits opened after fill has risen a couple of meters, have served the questionable purpose of checking themselves as recorders of overburden pressure increments. One can reason however, in average, that like effects associate with like causes. Therefore it suffices to examine experimental data collected (in the field of pavements) from tire pressures, analogous to pneumatic rollers.

Some published data are reproduced in Figs. 6 and 7, and require little comment since the experimental evidence fully confirms theoretical predictions. It is of interest to note the obvious increase of applied and transmitted pressures in changing from yielding to unyielding support (Fig. 7): in other words, with increasing number of passes and compaction, there is a gradual increase of rigity of support and also of the very pressures transmitted and absorbed (cf. Figs. 8a, 9a, 9b).

Another very important point is the much more significant PC gradient across the lift in sandy material than in clayey soil, and in dry compaction compared with wet compaction (Fig. 6b).

2.3 Schematic visualization of compaction hysteresis and bearing capacity limit of compactability in clayey soils

Fig. 8 configurates schematically the obvious mental model that emerges regarding absorption of compactive energy. In clay the compression-expansion hysteresis is well recognized to be very significant because of plate-like grains and change of structural arrangement. Note that aninteresting index of how much capillary negative pore pressure should develop in a given clay should be associated with the difference in the expansion behaviors on stress release (as the roller moves away) with and without access to free water (surround-

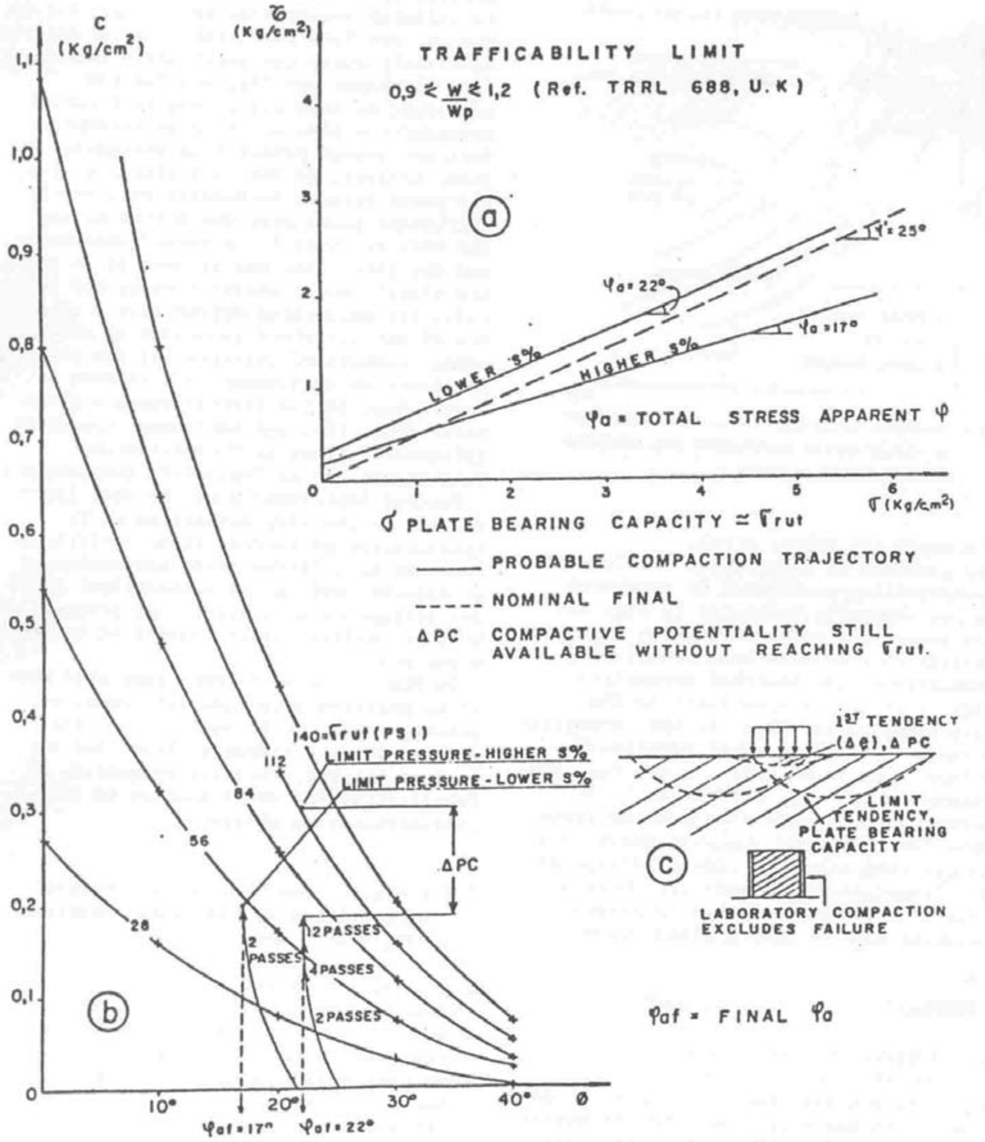


FIG. 11 PROGRESSIVE COMPRESSION - COMPACTION UP TO BEARING CAPACITY. TRAFFICABILITY LIMIT

possible "hang-up" of the cores) and the pressure data attributed to the field behaviors are well confirmed. Finally, it must be repeated that only one or few points of field observation are represented in each case because all of the other points gave almost exactly the same indications: the scatter in laboratory data is apparently great, but the consistency of field observed behavior was so very close as to be surprising (and to render impossible packing more curves into the same drawing).

The principal conclusions from these observations stand in surprisingly emphatic confirmation of some of the points discussed regarding the need for very significant revision of the routine test

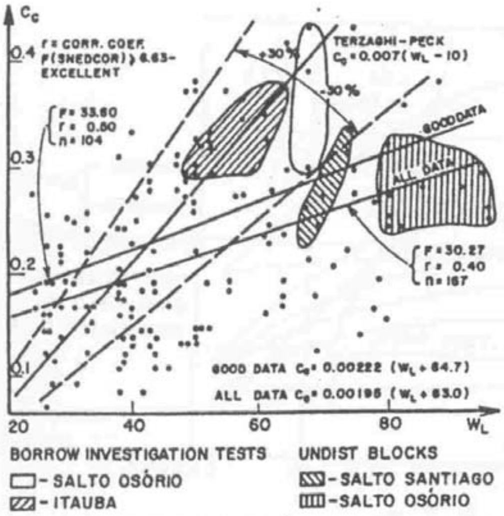


FIG. 12 REGRESSIONS OF C_c VS. W_L UNDISTURBED BLOCK SAMPLES

procedures and corresponding parameters.

Laboratory molded specimes would seem useless, both if tested in oedometer compression and in triaxial compression. One suggestion for the oedometer test might be to mold the specimens by compacting directly into and against the oedometer ring, incorporating into the specimen an initial lateral confining stress: it would require research and adjustment. The use of specimens cut from undisturbed block samples from the compacted fill offers some improvement: that is why in our design and construction experience with compacted earth dams we prefer to use the first few thousand cubic meters of actual placement and compaction as a field compaction test for field adjustments and for extraction of undisturbed block samples. One suggestion would be to resort more to field tests (plate load tests, duly conducted and interpreted, or pressuremeter tests, etc...) rather than so-called undisturbed block samples: thereby one approaches somewhat more the in situ condition that should retain residual stresses from compaction (de Mello, 1977).

Sophisticated laboratory tests on 4" diameter undisturbed specimens from block samples do offer some improvement (cf. the K_o tests and the anisotropic triaxial compression with σ_1/σ_3 ratio of 1.5). The predicted strains (and settlements would still be of the order of 2 to 3 times higher than the field behavior observed. It hardly seems worth the trouble: moreover there is no plausible theoretical reasoning in favour of such sophistication. Therefore, in other cases the comparison could be either less favourable or more so, purely by coincidence.

Note that the recompression strain values from oedometer tests on the undisturbed block samples come adequately close to representing the field behavior. Therefore, a technique may well be adjusted for using such simpler tests, but resorting to two, three, or more cycles of compression-decompression, before extracting the appropriate recompression E value (much as was done for building settlements on Stuttgart, cf. Schultze and collaborators).

One very remarkable observation is, of course, the fact that the clay compressibility is not, as was early feared, significantly greater than that of the clean sound dense basalt, compacted in 0.8 to 1.2m lifts.

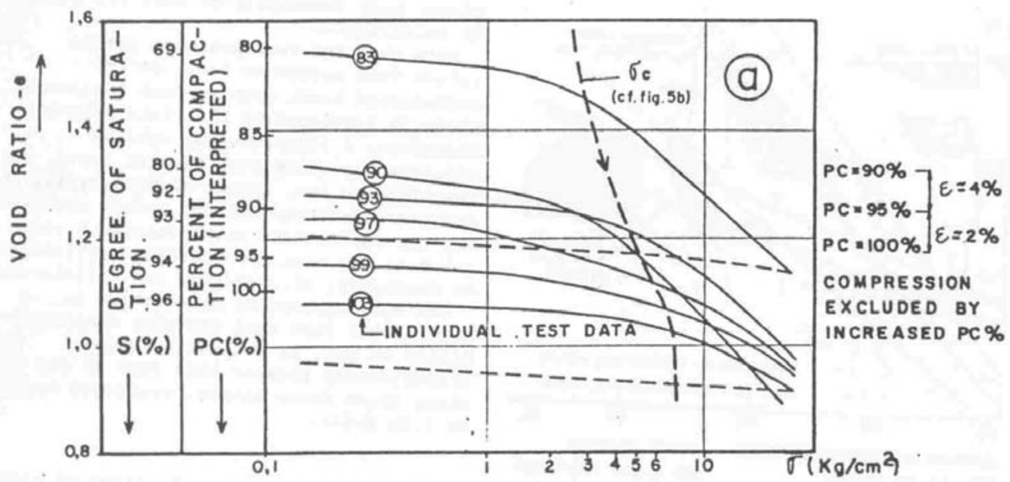
3.3 Field settlement and E values of clean well compacted angular basalt rockfill

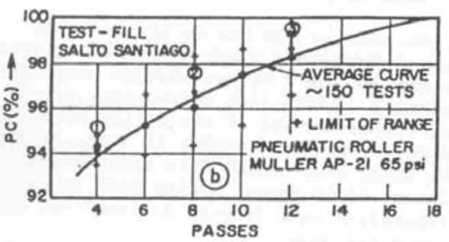
In the case of the Foz do Areia dam of record dimensions every care was exercised to improve the E values through compaction, watering, varied lift thicknesses etc... The limitation was of the weight of the vibratory roller (roughly 10 tons., static). The field compression data were so consistent as to appear having been faked: dispersions were frequently less than 2% around the mean.

Fig. 22 summarizes some of the typical settlement data which, once again very clearly demonstrate the precompression effect. If such a reasonable conclusion is really proven, for the cases of higher dams what would be required is to use heavier vibratory rollers with proportionally higher impact so as to increase the crushing of angular intergranular contacts during compaction.

Finally, in closing this presentation it is of interest to compare the behaviors of variations of E for the core and for the rockfill, with increasing σ. Firstly the difference is not at all one in the range of $E_{clay} = 80$ kg/cm^2 to $E_{rock} = 1000$ kg/cm^2, but much closer to similar. Secondly, due to more perceptible "jumps" in the early stages of rockfill compressions there is considerable initial scatter, even using secant E values (average from start): scatter would be much higher at start if employing $d_s/d\sigma$ values for E. Finally, beyond a certain loading the rockfill compressions become quite smooth, and, surprisingly, reach, in the cases presented, greater compressibilities than the very clay beyond a certain pressure (Fig. 23).

There is at present no means for





was to correlate with the liquid limit (Fig. 12). The interesting fact is that the correlation would differ considerably from those of sedimentary clays (Terzaghi-Peck, 1948), and principally so with "highly plastic" borrow materials. The liquid limit test measures the plasticity potentiality of a fully plasticized material. The fact is that the red porous basalt clays under consideration (and others similar) really occur in the borrow in granular conditions of hard clay nuclei, and the ω_L index becomes quite irrelevant. In other materials of wide grainsize distribution another obvious reason for discrepancies is the sieving out of grains or nuclei above the # 40 sieve for the ω_L test, whereas in compaction tests the sieving is on the # 4 sieve, and in field compaction much bigger nuclei are incorporated depending on the equipment used, and the in situ conditions in the borrow.

A very interesting result is obtained by recognition that the Proctor compaction test itself is an appropriate index for classification of soil types. Excellent coefficients of correlation were obtained (Fig. 13) for estimation of C_c values on undisturbed block sample oedometer tests on the basis of Proctor compaction maximum

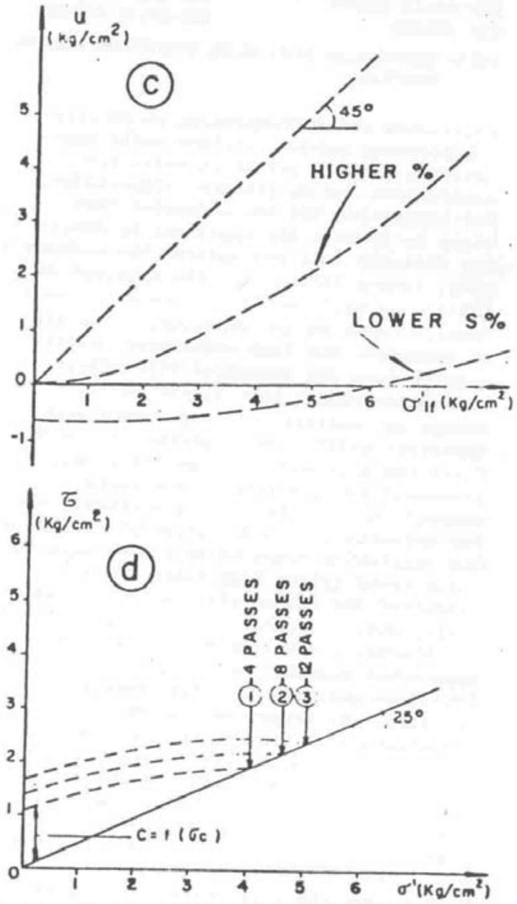


FIG.9 COMPACTION SEQUENCE WITH INCREASED PASSES, ESTIMATED CHANGE OF STRENGTH ENVELOPE

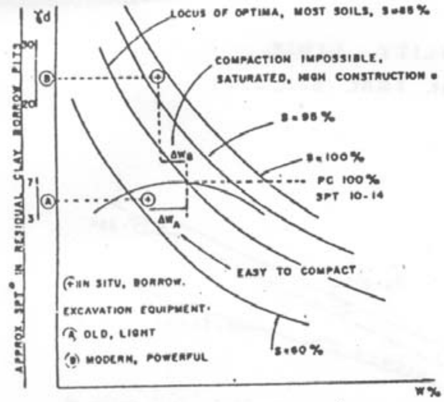


FIG.10 SCHEMATIC EXPLANATION OF WHY THE ΔW% TEST OF BORROW PITS IS INSUFFICIENT FOR INDICATING COMPACTABILITY OF CLAYS

dry density (de Mello, 1977).

The problems of comparative E values of compressibility-settlement in earthrock dams are summarily indicated in Fig. 14. Below compaction preconsolidation pressures materials tend to have been practically homogenized by the absorbed compactive energy: differences accentuate in the virgin compression range. In the recompression range the "compressible materials" (exclude dense sands, etc..., cf. Fig. 8b) E values are not very different. It is important to recognize this problem since simple finite element analyses seduce one into the impressions of linear extrapolations. Precedents of satisfactory behavior of low dams cannot be lightly extrapolated to similar dams of much greater heights.

3. COMPARATIVE BEHAVIORS OF DAMS

Fig. 15 gives the basic data on cross-sections of three earth rock dams employing the very clayey red porous clay compacted core, and the very sound angular quarry basalt compacted rockfill. Fig. 16 gives the corresponding crosssection of the concrete-face rock-fill dam of much greater height employing exactly similar compacted rockfill.

3.1 Need to compute overburden pressures including some Influence factor $I \lessgtr 1.00$, and interest in plotting on log σ scale

Practically all publications on behavior of embankment dams have automatically assumed that increments of overburden stress γz are applied and transmitted as if pertaining to infinite loaded area conditions, I=1.00, and yet are limited strictly to the material vertically above the point under consideration. If slopes are flat and the dam construction does not incorporate varied construction phasing, such an assumption does not become patently unreasonable. In fact, however, as the fill rises, a given mid-point between vertically positioned settlement gages near the bottom of the dam only receives $I\gamma z$ pressure increments, and the least that can be done is to use the elastic model charts (Poulos and Davis, 1974) for estimating approximate I values. One of the illusions generated by such a wrong practice of interpreting and plotting construction settlement data is that at some moment $\Delta\sigma$ has stopped above a given point (Fig. 17), and settlement continues (discussed either as "consolidation settlements" or as "secondary compressions").

Further improvements may be made iteratively in the very assumption of γz (presupposes no lateral shear redistributions due to differentiated tendencies to settlement) and in the mathematical model for extraction of I values. At present, however, suffice it to correct an evidently wrong practice.

In Fig. 17 it is further indicated that it is preferable to plot settlement vs. pressure data in the semilog plot that was adopted for the oedometer test, for the express purpose, entirely pragmatic, of facilitating the determination of the preconsolidation pressure.

3.2 Field settlement behavior compared with laboratory test data, compacted very clayey core

Figs. 18, 19, 20 and 21 summarize the comparative information on compressive vertical strains as derived from laboratory tests, both routine and sophisticated, and from settlement observation of the construction settlements.

Firstly it must be mentioned that for the purpose of approximation with current routines, the field data on σ have not been corrected with regard to I. Fig. 22 summarizes, for comparison, typical data on some settlement gages, presented both with respect to γz and with respect to the nominal elastic $I\gamma z$: the principal consequence is that the log $(I\gamma z)$ graph accentuates more definitely the apparent nominal preconsolidation pressure.

Secondly, it must be mentioned that overburden stresses were measured in the core (because of fears of redistributions due to incompatible deformabilities and

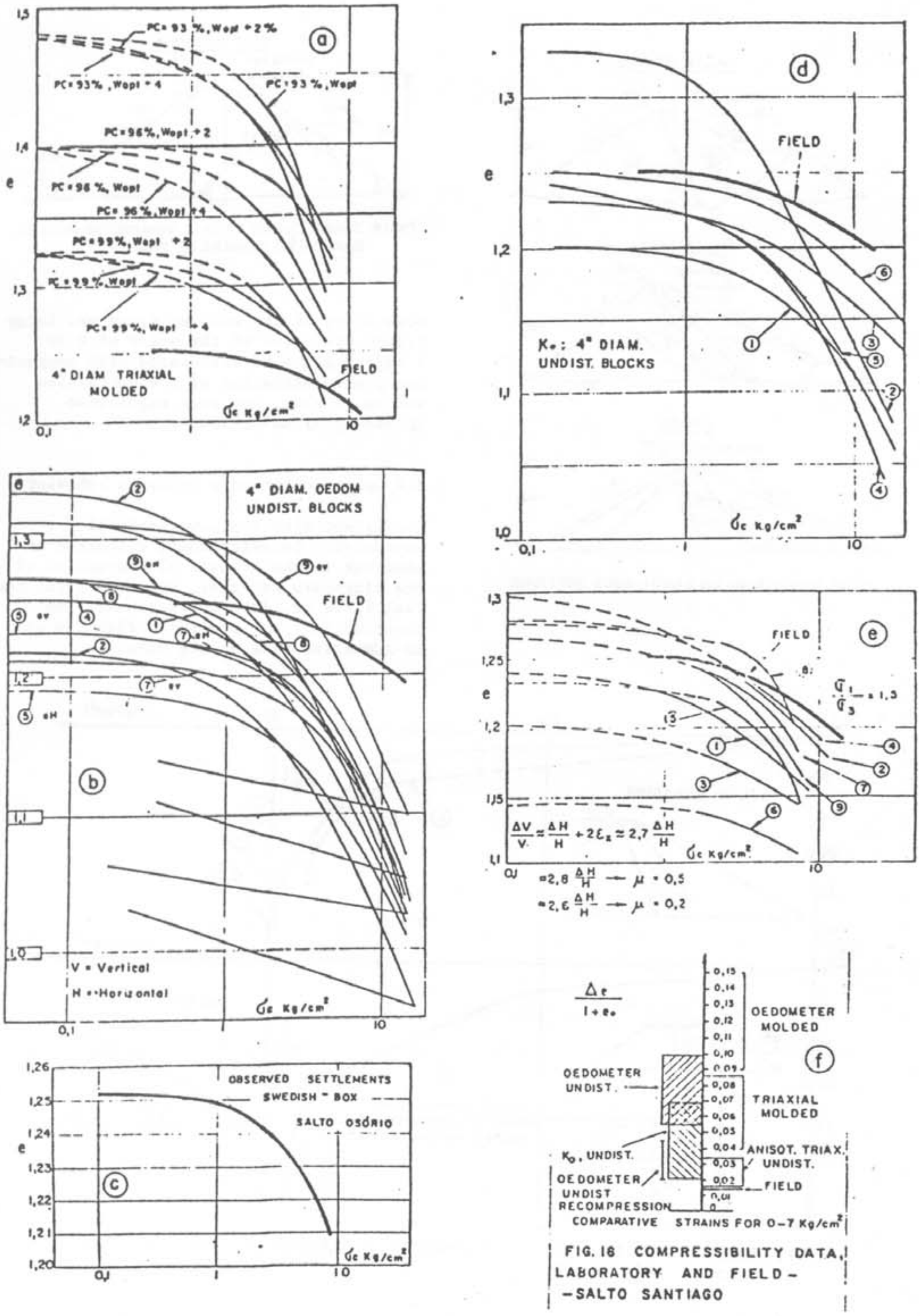


FIG. 16 COMPRESSIBILITY DATA, LABORATORY AND FIELD - SALTO SANTIAGO

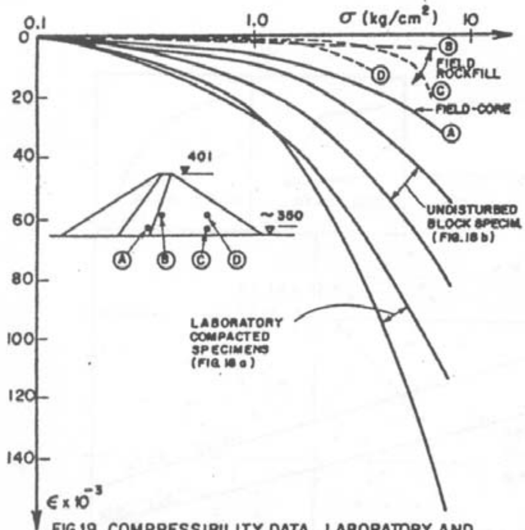


FIG.19 COMPRESSIBILITY DATA, LABORATORY AND FIELD - SALTO OSORIO

Both the early USBR (1930-1955) field observed data, hampered by a big consistent error of high initial u values due to soaking around the cell, and the indications from near-saturated borrow pits subjected more to remolding than to compaction (cf. some European and British experience), have spread an impression of great problems due to construction pore pressures. In fact, the limits of trafficability quite exclude such a possibility. Truly it has been and continues to be very difficult to measure negative pore pressures, that are inexorable right after compaction of a clayey material (duly unsaturated): it is difficult in the laboratory, and much more so in the field. In field observations even with the best of constant volume cells, and respective installations, the early measurements tend to be erratic "around zero": only beyond a certain overburden pressure do the readings begin to develop the consistent trend of concave-upward curve of increasing u and increasing rate of change of u with σ. The best way to estimate initial conditions in situ would be by extrapolating backwards, with due cognizance of the varying expansivity with stress release as the roller moves away.

The typical detailed data of trends of u vs. σ from the Salto Osorio field observations have been presented (de Mello, 1975): positive pressures only begin to build up, at a small rate, beyond an overburden pressure of the order of 4 kg/cm^2. Moreover, as summarized in Figs. 24 and 25, all the test data are most unrealistically pessimistic. The impression that a very clayey material should give high construction u values is obviously unrealistic. One should reason on the basis of the "equivalent suction" values necessary to give the same trafficability, in all materials: with a low ϕ' associated with a very clayey material, there has to be a compensation of a high "equivalent suction".

In a separate paper we shall discuss how inappropriate it is to employ such ratios as $\bar{B}$ or r_u, wherein early values "explode to infinity" merely because σ begins from zero, and also how unnecessary, and detrimental to economy of shallow circle or low embankment analyses, it has been to employ a straight line u vs. σ variation starting from the origin.

4. SUMMARY

4.1 The experience cycle in the cases configurated demonstrates that conventional classification index tests do not lead to any valid predictions of behavior.

4.2 Compaction behavior of clayey materials depends very much on the condition of the clay chunks and/or nuclei in situ, in the borrow, and principally on S% and porosimetry (frequency of macropores). Conventional compaction indices PC% and Δω% may often lead to grave errors of planning of plant and of prediction of behavior.

4.3 Both clays and sound angular clean rockfills present definite evidence of nominal preconsolidation pressures corresponding to the compactive energy absorbed.

4.4 For the estimation of settlement behavior of compacted clay fills one should resort to adjusted field and laboratory tests, pertaining to field compacted volumes. Statistical regressions are offered for σ_c and C_c based on undisturbed block samples. The best classification index appears to be the Proctor maximum dry density.

4.5 Field compressibilities are much smaller than those of the regressions mentioned in 4.4, probably because of residual stresses from compaction, that benefit in situ conditions. Recompression curve settlements may be an avenue for

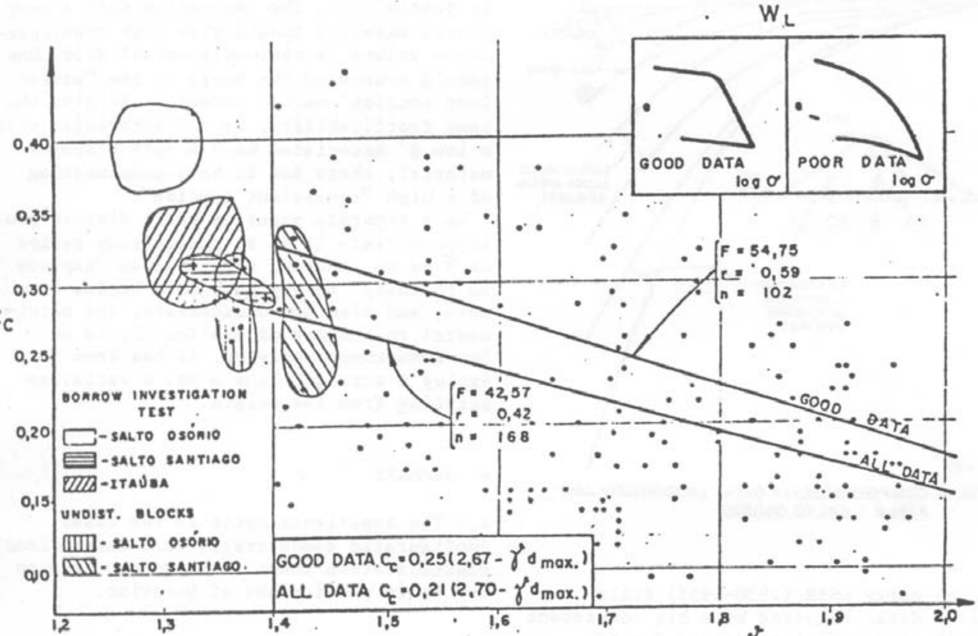
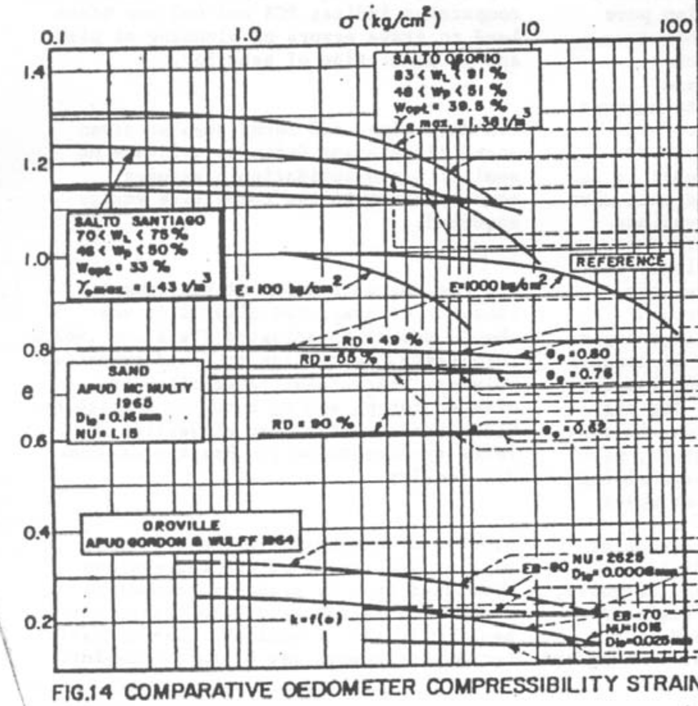


FIG.13 REGRESSION Cc vs $\gamma_{d\,max}$, SAME DATA



OEDOMETER % STRAIN $\epsilon = \frac{\Delta\sigma}{(1+e_o)E}$ kg/cm^2							
COMPRESSION						RECOMPR.	
σ 0.5-4.0 kg/cm^2		4.0-10.0 kg/cm^2		4.0-20.0 kg/cm^2		0.5-10.0 kg/cm^2	
ϵ %	E	ϵ %	E	ϵ %	E	ϵ %	E
3.03	120						
		3.79	160				
				8.01	210		
						1.96	460
2.47	141						
		3.69	163				
				8.76	183		
0.78	450					2.47	386
		0.44	1360				
				0.61	2600		
0.46	780						
		0.43	1400				
				0.91	1760		
						0.14	6890
0.25	1400						
		0.06	10000				
				0.46	5600		
						0.15	7000
2.41	150						
		2.11	280				
				4.21	380		
						1.53	620
2.70	130						
		2.69	230				
				4.77	340	1.19	800

FIG.14 COMPARATIVE OEDOMETER COMPRESSIBILITY STRAINS IN WIDELY DIFFERENT MATERIALS

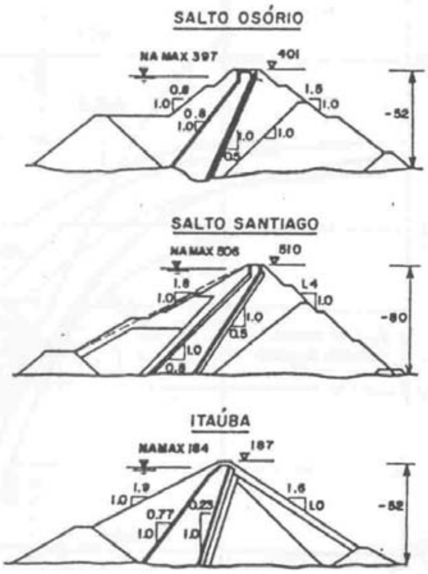


FIG.15 THREE SIMILAR EARTH-ROCK SECTIONS

FIG.16 SOUND CLEAN BASALT COMPACTED ROCKFILL DAM WITH CONCRETE FACE

evaluating if the smaller Erock vs. Eclay beyond pressures of the order of 6 to 7 kg/cm^2 might be associated with compaction preconsolidation or precompression values of about the same magnitudes (possibly by coincidence).

3.4 Construction pore pressure behavior

Consistent with the compressibility - expansivity behavior above discussed as inherent in the process of compaction of the clay-core material, one also finds the real field vs. laboratory behavior on construction pore pressure. Figs. 24 and 25 summarize some of the data.

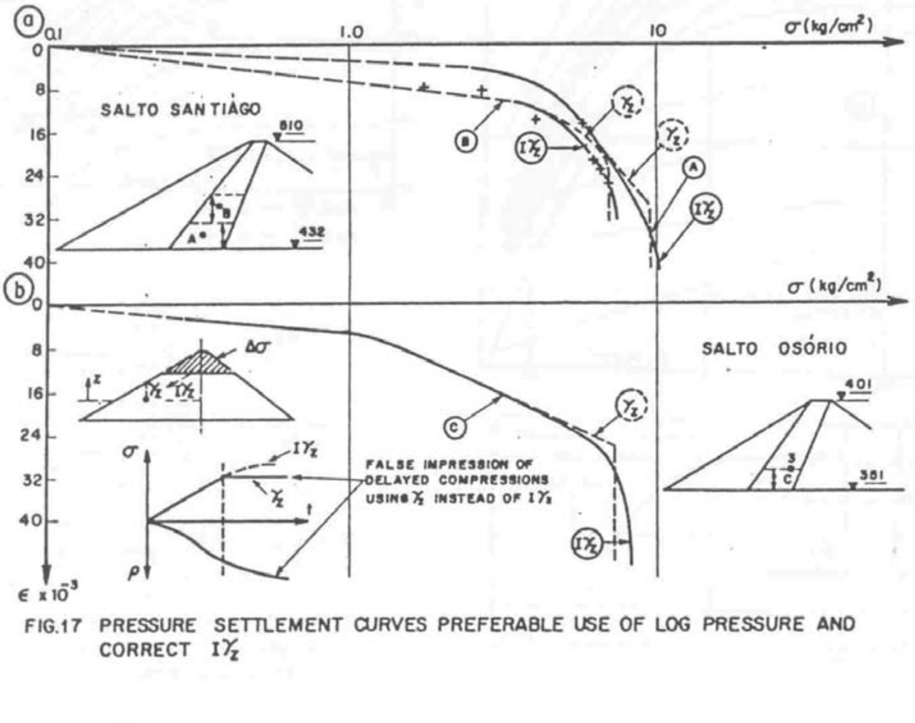
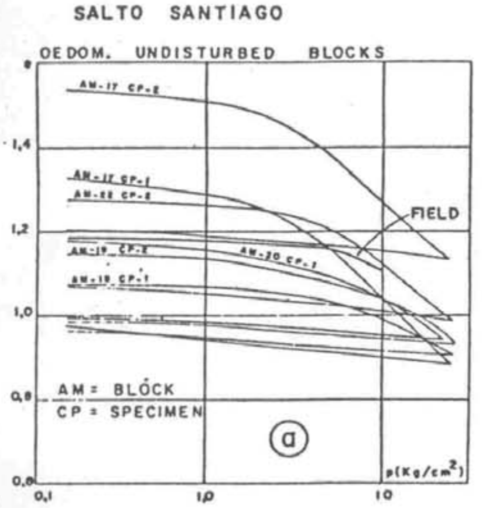
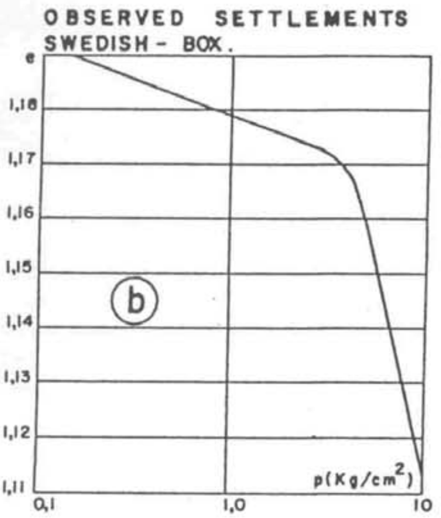
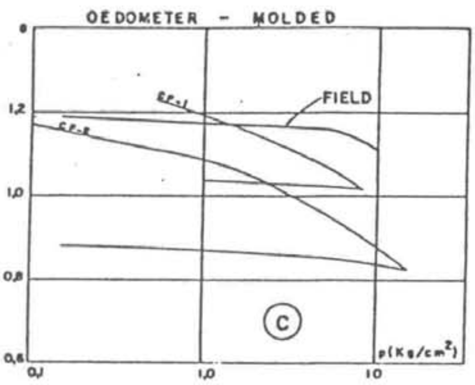


FIG.17 PRESSURE SETTLEMENT CURVES PREFERABLE USE OF LOG PRESSURE AND CORRECT $I\gamma_z$







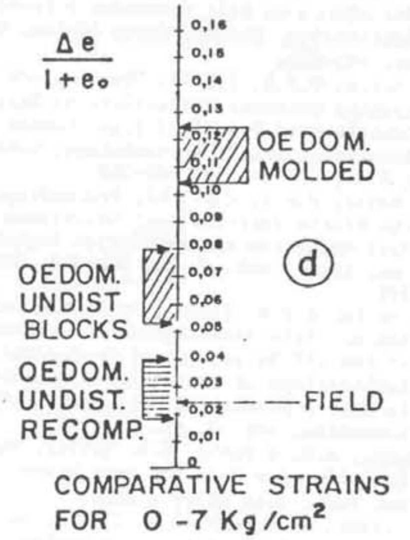


FIG.20 COMPRESSIBILITY DATA, LABORATORY AND FIELD — SALTO SANTIAGO

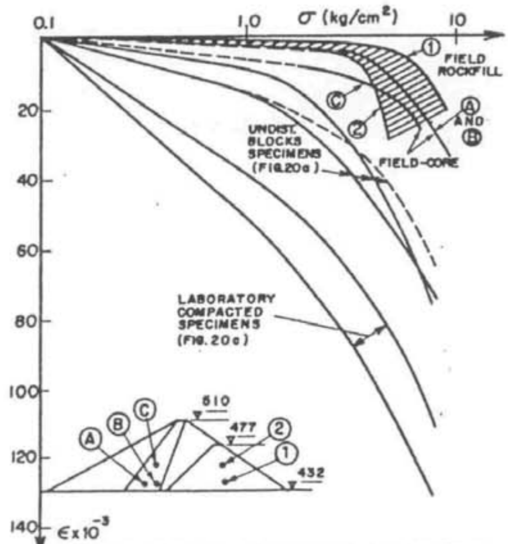


FIG.21 COMPRESSIBILITY DATA, LABORATORY AND FIELD - SALTO SANTIAGO

suitable adjustment between prediction and observation.

4.6 The guarantee of trafficability for modern heavy earthmoving and compaction

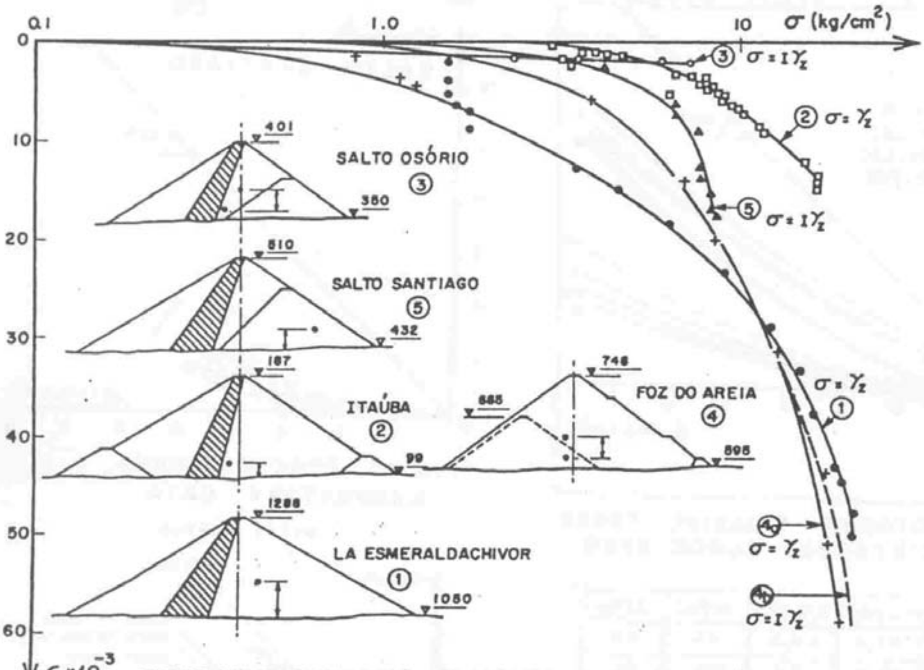


FIG. 22 COMPRESSIBILITIES AND NOMINAL PRECOMPRESSIONS OF COMPACTED ROCKFILLS

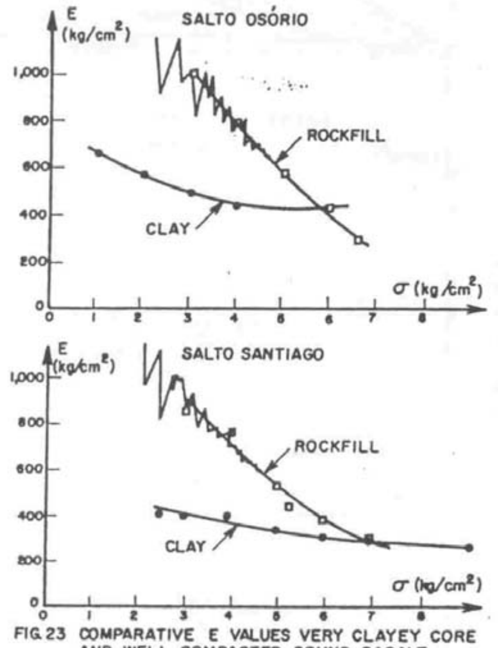


FIG. 23 COMPARATIVE E VALUES VERY CLAYEY CORE AND WELL COMPACTED SOUND BASALT

equipment is generally a sufficient guarantee that there will be no problem from construction pore pressures.

4.7 Many of the automatically accepted dictates of testing, parameters, and computations are herein shown to require rather significant revision. Precedents will not be sufficient to guarantee success if we are called to extrapolate, and herewith recognize that the mental models for such analysis and possible extrapolation are not really valid.

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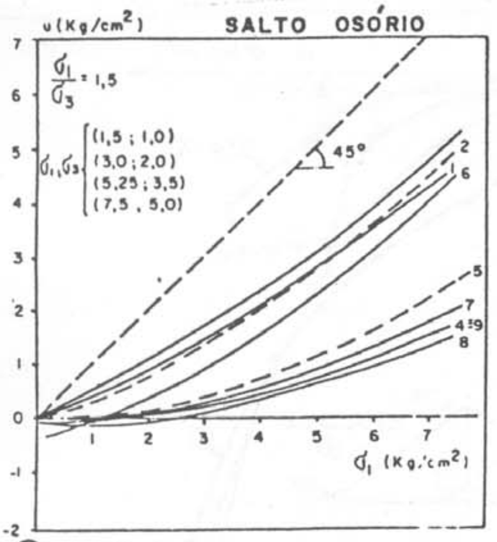
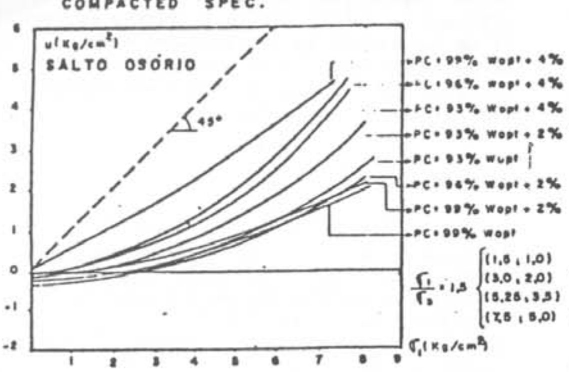


(a) ANISOTROPIC TRIAXIAL TESTS UNDISTURBED BLOCK SPEC.

AM	PC(%)	ΔW(%)	W(%)	S(%)
1	94,4	+ 4,3	39	94
2	95,4	+ 3,3	41	96
4	100,4	+ 2,2	39	93
5	100	- 0,5	38	93
6	101	+ 0,9	37	97
7	99,8	+ 0,1	36	83
8	99,2	+ 1,0	38	91
9	97,8	+ 1,1	39	92

(b) ANISOTROPIC TRIAXIAL TESTS LABORATORY COMPACTED SPEC.



(c) UNDIST. BLOCKS TRIAXIAL CU

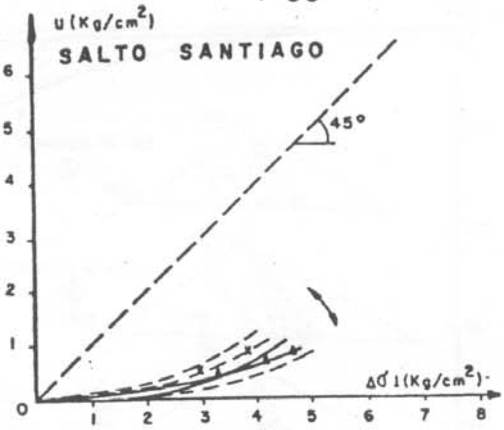


FIG. 24 POREPRESSURES. LABORATORY DATA

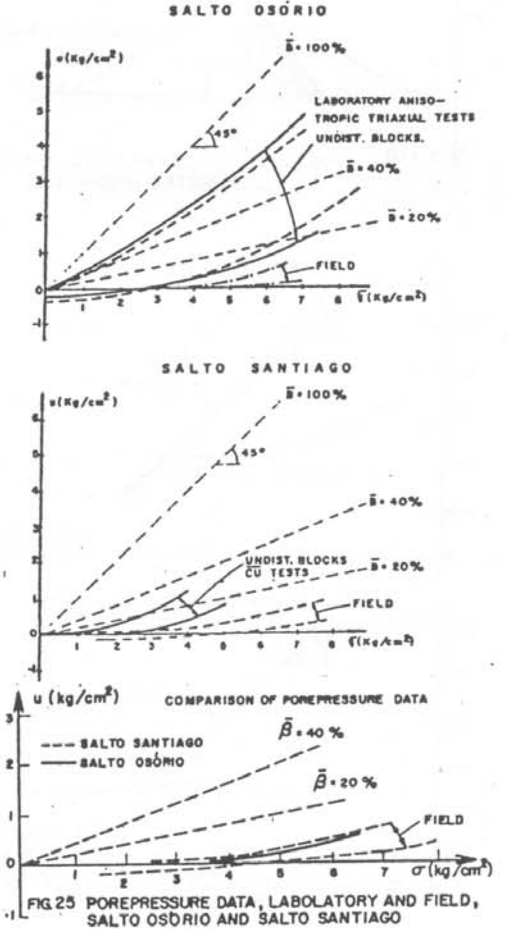


FIG. 25 POREPRESSURE DATA, LABOLATORY AND FIELD, SALTO OSÓRIO AND SALTO SANTIAGO

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