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Behavior of Two Big Rockfill Dams, and Design Aims

V. F. B. de Mello
ISSMFE

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V.F.B. de Mello
Professor, D.Sc., Consulting Engineer, President ISSMFE

SYNOPSIS The case histories of two major compacted embankment dams are analysed with regard to the problem most amenable to reasonings and computations of statistics of averages, the problem of deformations, discussed regarding specific laboratory testing compared with use of generalized correlations. Improved mental models for predictions are proposed. In the earthcore-compacted rockfill section the problem of crest cracking suggests the interest in a significant change of zoned section in the topmost stretch. For the concrete-face rockfill dam it is suggested that one needs a significant revision of instrumentation and monitoring orientation. Both dams behaved extraordinarily well on questions of consequence and served to show that unfavourable observations, if too indirect, lead to no benefit, but may sometimes prove the inexistence of the consequential misbehavior to be guarded against.

1. INTRODUCTION

Over the past few years I have deepened my convictions, and repeatedly emphasized, that the basic concepts of design of important civil engineering structures comprise five steps summarizable as follows:-

1) firstly seek and adopt for the desired functional need a type of structure that physically precludes behaviors of significance subject to the statistics of extreme values, truly unquantifiable, especially if and when capable of progressive accelerative degeneration towards failure (de Mello, 1977). Phenomena such as the weakest link tension failure case should, in a good design, be precluded; not by increasing nominal computed factors of safety, which always generate extremely uneconomical conditions and often are truly illusory, but rather by choice of change of physical universe, so as to work with other statistical histograms of definable limiting quantification.

2) having chosen the functional model, whose significant behaviors are dictated by the statistics of averages, engineering acquires the right to meaningful calculations, and a four-pronged attack occupies the gross of apparent energies of engineering workers:-

(2.1) the prediction of what is likely to be the behavior is associated with the statistical average behavior, equated, adjusted, and extrapolated: the ultimate aim would be to be able to predict "exactly"; adjustments and extrapolations require "laws", that is, repetitive responses to meaningful index parameters;

(2.2) however, for many inexorable realistic reasons, the engineering design decision is based on estimates of the unfavourable "probability limits" (let us say, the 90% confidence bands) around the average, and on the guarantee that in all probability the threshold of undesirable behavior will not be crossed. Most often the self-same material must satisfy two opposite limits: for instance, a compacted clay core must ensure that it will not suffer differential settlements higher than a certain value, whereupon the design decision is based on the upper confidence limit of compressibilities not being exceeded; the self-same compacted clay core may, in certain zones, be sought to ensure that brittleness and rigidity are minimized, and, therefore, for such zones the compressibility (often considered to accompany plastic deformability) may be specified not to fall below the lower confidence limit. Thus, the engineering decisions may be said to be based on prediction of what will most probably not happen, rather than on the direct aim at computationally reaching more accurate prediction of what will presumably happen in reality (deterministic).

(2.3) the basic needs for design computations are, therefore, to improve the predictability on average parameters and adjusted methods of computation, and to narrow the width of our confidence bands, as rapidly and efficiently as possible.

For any forthcoming job this is achieved on the basis of well digested data from monitored projects; and for any given project under construction, by rapidly adjusted iterative decisions by Bayesian prior-to-posterior probability adjustments from greater intensity testing and monitoring right at the bottom, for judicious extrapolation to the nevralgic conditions when the job is crowned and subjected to first-filling.

(2.4) finally, it recognizedly often happens that cumulative experience ("common sense" being a cumulative intuition developed by Bayesian "natural selection") on complex lumped parameters of prototype to prototype comparison will yield better design decisions and behavior estimates (Hynes and Vanmarke, 1977), than a specific new set of test data and/or design computations.

3) thus it is that the analysis of case-histories acquires significance not only with regard to the sensational attention to patent failures (indicating physical models to be guarded against) but also with regard to the large silent majority of cases of varying degrees of misbehavior which will permit quantifying the decision thresholds.

The intent herein is to consider the case histories of two recent dams of about 160m in height (Figs. 1, 2) and impressive data. It may be accepted that both the dam sections are of chosen physical models well conceived to avoid sensational failure, and therefore it is understandable that attention has concentrated on problems of deformations which have in various projects generated varying degrees of misbehavior. Deformations result from integrated effects and are therefore well included in the group dominated by the statistics of averages. The paper concentrates on such problems, both for predictions and design decisions, and as leading to suggested revision for future cases.

Note that both cases also supplied very interesting data regarding grouting treatments and efficiencies in sealing horizons of open--fractured rock: and it is recognized that the foundation treatment problem is another one of the dominant ones in such dams; however, for convenience in concentrating on a single message, the paper will not touch on these facets, which were most successfully handled.

2. BASIC DATA ON SOME COMPACTED EARTH DAMS

Brazilian experience on test compressibilities and monitored compressibility settlements of dams ranging between 50 and 160m has often enough been published case by case, generally showing that observed settlements have been significantly smaller than those predicted. Table I summarizes the principal data on the dams under consideration and settlement predictions based on oedometer tests. It may be remarked that the principal concern arises from the fact that the ratios of observed: predicted (cf. Fig. 19) or differences of predicted minus

TABLE I

DAM	HEIGHT (m) MÁX./INSTRUM SECTION	SOIL TYPE	GEOLOGIC ORIGIN	SOIL CHARACTERISTICS W_L (%)	I_P (%)	ρ_s (g/cm³)	STANDARD PROCTOR γ_d máx (g/cm³)	STANDARD PROCTOR W_{opt} (%)	COMPACTION (FIELD) PC (%)	* ΔW (%) (W-Wopt)	TEST	END-CONSTR SETTLEM. (cm) OBSERV.	PREDICT. OEDOM.	KEY IN FIGS. (TESTS) UNDIST. BLOCKS	MOLDED
SALTO OSÓRIO	56/52	RED SILTY CLAY	BASALT DECOMP.	70 - 85	25 - 35	2,9 - 3,0	1,3 - 1,4	34 - 38	99 - 101 100	-1 / +1	HILF	41	54	○	●
SALTO SANTIAGO	80/80	RED SILTY CLAY	BASALT DECOMP.	62 - 72	17 - 22	2,9 - 3,0	1,35 - 1,45	30 - 34	97 - 98	0 / +1	HILF	81	244	△	▲
PARAIBUNA	94/61	SANDY CLAY	BIOTITE GNEISS RESIDUAL SOIL	65 - 73	30 - 35	2,72 - 2,83	1,5 - 1,6	23 - 26	99	-1 / +0,5	STD PROCTOR	30	72	—	◒
		SANDY SILT		44 - 51	NP - 23	2,69 - 2,79	1,6 - 1,7	16 - 19							
PARAITINGA	104/64	SANDY CLAY	BIOTITE GNEISS RESIDUAL SOIL	53 - 87	23 - 42	2,78 - 2,86	1,42 - 1,63	26 - 29	99,2	-0,5 / +1,5	STD PROCTOR	47	79	□	■
		SANDY SILT		34 - 48	NP - 19	2,69 - 2,79	1,64 - 1,80	14 - 20		-1 / +0,5					
CAPIVARA	60/60	SANDY SILTY CLAY	BASALT DECOMP.	40 - 50	15 - 20	2,83 - 2,92	1,55 - 1,68	20 - 25	97 - 98	+1 / +2,0	STD PROCTOR	57	98	◩	—
EMBORCAÇÃO	150/150	SANDY SILTY CLAY	BASALT DECOMP. AND GRANITE--GNEISS DECOMP.	42 - 52	15 - 25	2,6 - 2,8	1,55 - 1,75	20 - 25	101 - 104	-1,5 / 0	HILF	180	285	▽	▼
ÁGUA VERMELHA	63/47	SANDY SILTY CLAY	BASALT COLLUVIAL	38 - 45	13 - 19	2,86 - 2,95	1,6 - 1,8	17 - 21	98 - 100 99,7	-0,6 / +1	STD PROCTOR	10	45	◊	♦
ITAÚBA	93/93	SILTY CLAY	BASALT DECOMP.	50 - 80	30 - 40	2,7 - 2,85	1,3 - 1,45	28 - 33	97 - 100 98,5	+1 / +2	HILF	92	240	—	x
PASSO REAL	54/54	SILTY CLAY AND CLAYEY SILT	BASALT DECOMP.	50 - 70	10 - 22	2,65 - 2,80	1,3 - 1,4	32 - 34	97 - 98	-1 / +2	HILF	40	99	—	*
MARIMBONDO	70	SILTY CLAY	BASALT DECOMP.	29 - 35	12 - 20	2,80	1,83 - 1,96	12 - 15	—	—	—	—	—	—	◓
PEDRA DO CAVALO	130/95	SANDY SILTY CLAY	GRANITE-GNEISS DECOMP. AND BARREIRAS FORMATION	32 - 40	12 - 22	2,65 - 2,75	1,8 - 1,9	12 - 16	98 - 101	-2 / +0,1	HILF + HILF - PROCTOR	60	86	⊕	◕
SAO SIMÃO	127/64	CLAYEY SAND	BASALT DECOMP.	25 - 40	10 - 22	2,7 - 2,85	1,8 - 2,0	11 - 15	100 - 102	-1,2 / -0,2	HILF	28	31	—	+
S. SANTIAGO (AUXILIAR DIKE)	60/60	SILTY CLAY	BASALT DECOMP.	62 - 72	17 - 22	2,9 - 3,0	1,35 - 1,45	33 - 38	97 - 98	+1,5 / +2,6	HILF	77	176	—	—
		CLAYEY SILT		55 - 64	17 - 20	2,88 - 2,95	1,35 - 1,45	33 - 38	97 - 98	+2,0 / +2,6	HILF				

* OBSERV. - SPECS CONVENTIONALLY WRITTEN AS < ΔW < PRESENTLY PREFERRED AS $0{,}95 < \frac{W_{COMP}}{W_{OPT}} < 1{,}05$ (ex.)

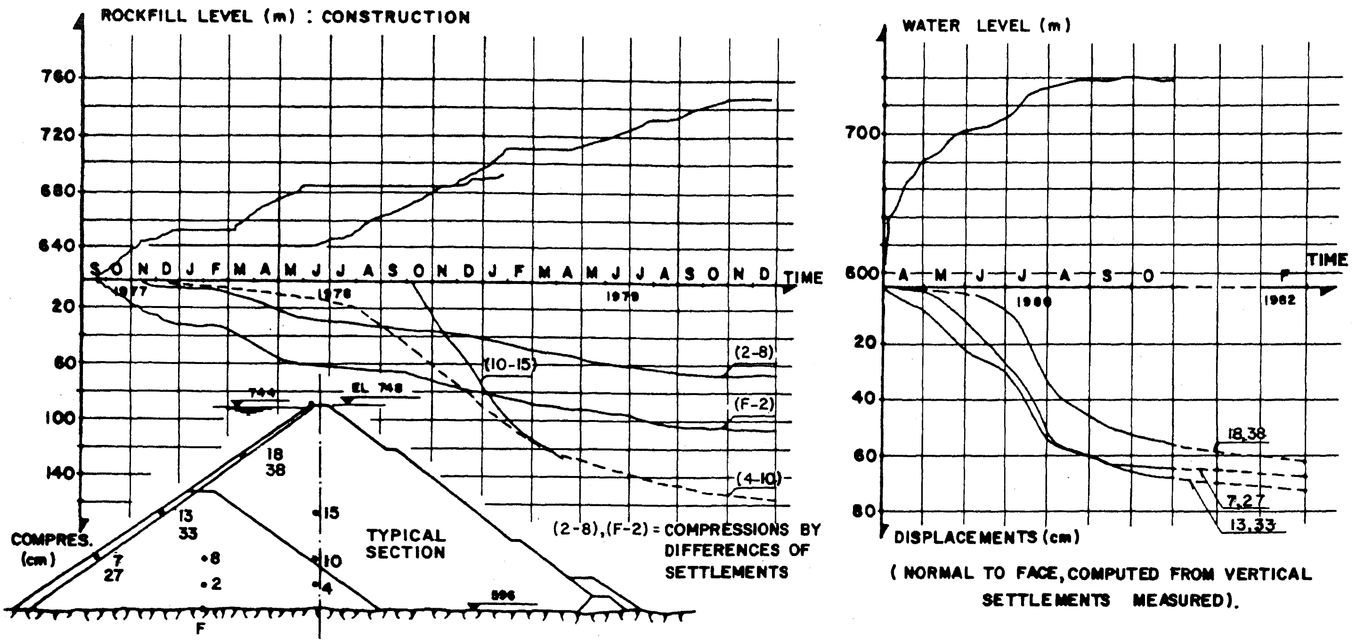


Fig. 1 - Summary Data, Foz do Areia Dam

observed settlements accentuate in cases of greater settlements, thus indicating errors of trends and extrapolation, in contrast to erratic errors (statistical dispersions); since differential settlements have been associated with generating cracking misbehavior at the vulnerable top, the noted trend is of definite importance to design and performance.

The first part of the present study is thus directed towards the statistical digestion of laboratory and field data on compressibilities for the purpose of improving the basis for design estimates.

In previous papers (de Mello, 1977; 1980) it has been emphasized that the most fruitful mental model for interpreting behaviors of compacted clayey soils is to consider compaction as equivalent to producing a certain preconsolidation pressure $\sigma'p$. Such a preconsolidation pressure should be directly related to the percent

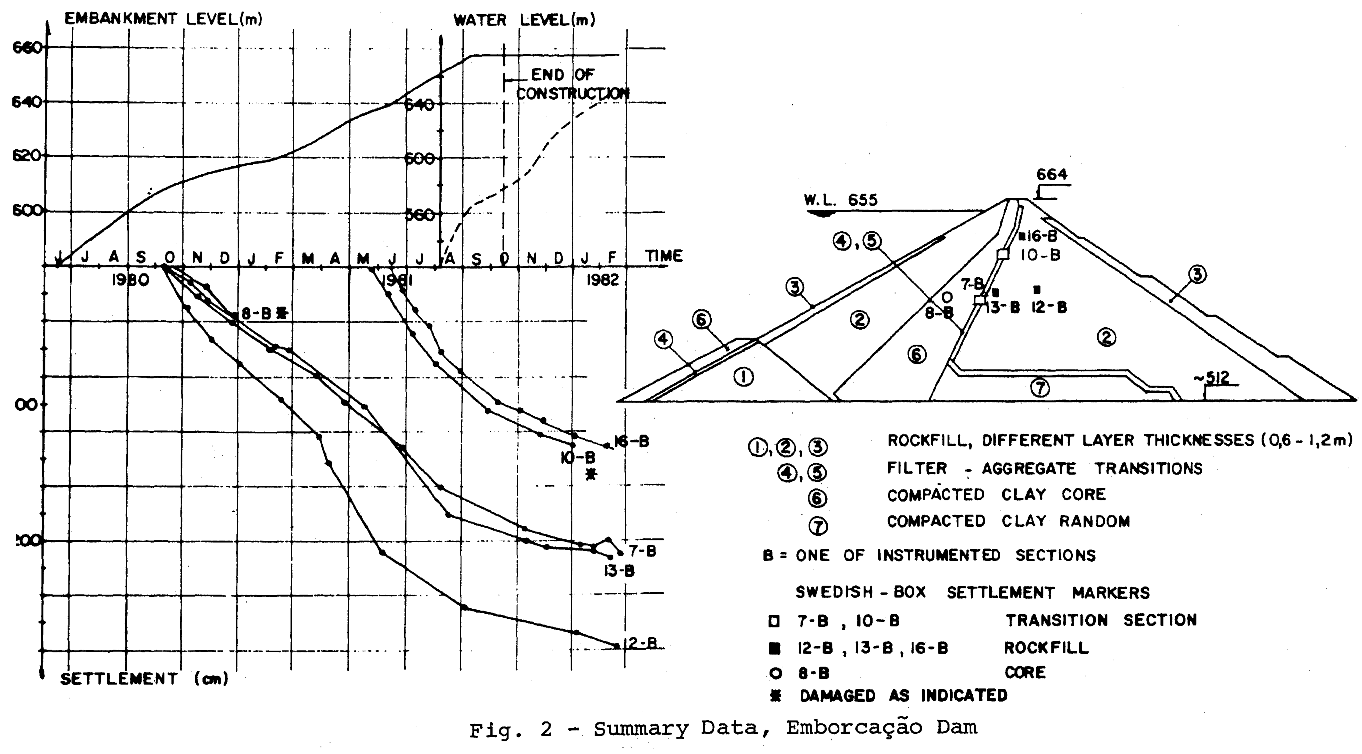


Fig. 2 - Summary Data, Emborcação Dam

compaction PC%, and, presumably, within the narrow ranges of PC% in which a given project works, beyond the σ'p value the nominal virgin compression index Cc would be essentially constant. Within such idealized preliminary reasonings the following analyses and improvements are submitted. Some second-order adjustments are discussed as appear justified.

3. STATISTICAL CORRELATIONS INVESTIGATED

The available data have been analysed separately in soil groups as distinguished by the respective maximum dry densities. In previous papers (de Mello, 1977; 1980) it has been shown that the attempt te relate soil types to Atterberg limits as appropriate index tests would be much less fruitful. Fig. 3 reproduces generalized data on compression indices Cc against a background of hypothetical correlation with liquid limits; one should note that the more "plastic soils" (higher liquid limits) have systematically given much smaller settlements than would be inferred from routine classifications evolved from saturated sediments. The reasoning concerning compaction is that on the one hand the absorption of precompression effects by volumetric vs. shear deformations varies as one moves from sandier to more clayey behaviors: and, on the other hand, the capacity to retain such precompression benefits also varies as a function of the expansivities on release of applied stress, and retentions of pore suctions that avoid heave (themselves varying from sandier to clayey soils).

3.1. Compaction Preconsolidation Pressures

In Fig. 4 we begin by analysing the most clayey soils (γdmax < 1,5 t/m^3) in which the above-mentioned hypothesis would seem most applicable. All the available data were used for a best fit semilog linear statistical regression. It is indiscriminate and consequently spurious: firstly because there was no attempt to employ a theoretically suggested reasoning for the inclination of the semilog linear statistical regression; secondly, because there was purposely no attempt to distinguish between specimens molded in the laboratory and those trimmed from undisturbed block samples from the compacted fill, thus disdaining the fundamental tenet that everything is different unless and until proven reasonably similar.

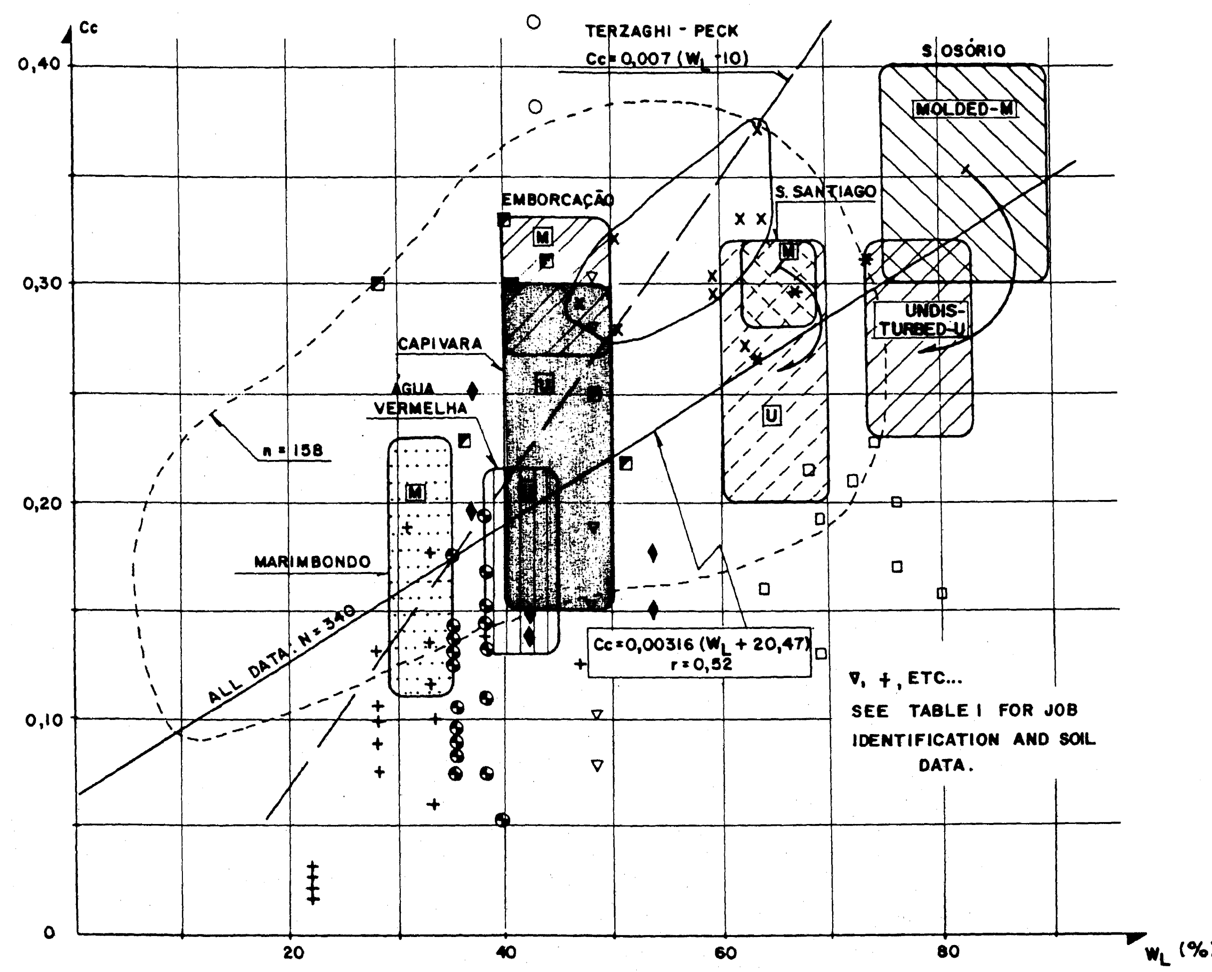


Fig. 3 - Routine Attempt to Infer Cc Compressibility from Liquid Limits

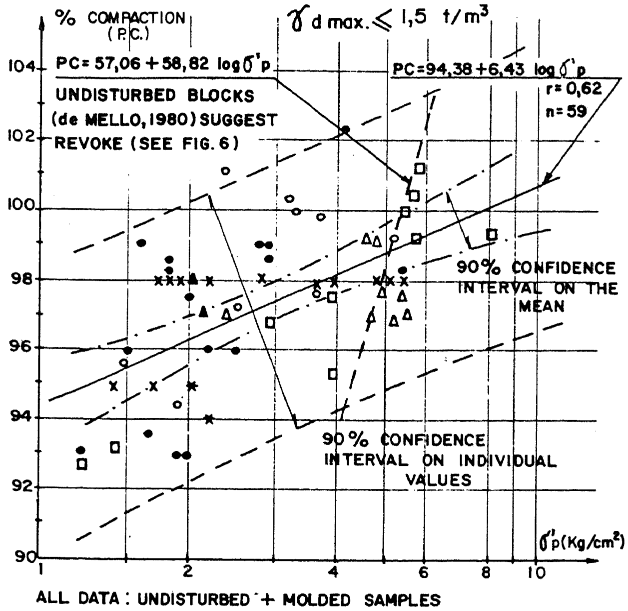


Fig. 4 - Statistics for σ'p, Most Clayey Soils

The figure is submitted on purpose, showing firstly the widely different regressions that can be obtained. Secondly, it is intended to emphasize the big distinction between confidence limits on averages and those on "single values". I submit the admonition that in order for mathematical statistics to lead to fruitful results, it is indispensable to use all possible indications that separate deterministically distinct universes, and also to use available "laws of behavior" to preestablish the type of appropriate equation for desired best fit. On the same figure, I also show a regression equation extracted from fewer data as published by de Mello (1980), which is herein considered revokable.

Often we are led to insist on tests for a specific soil and project and to adopt conservative parameters based on specific test results. Such a directive would suggest the attaching of special importance to the confidence band on single values. In cases such as that of compressibility of porous granulates, especially as reflected by specimens of truly small dimensions, it is evident that individual results may be subject to a wide scatter that would never be reflected in bigger masses of the prototype. Thus, experience from many projects may lead to much more valid predictions: in statistical terms one should place much more faith in confidence bands on the averages.

Theoretical idealizations are required for arriving at more meaningful best fit regressions, and moreover, a judicious decision must be made regarding applicability or not of dispersions of averages, or of individual values. Therefore one should conclude that although each additional case-history tested for design, and monitored for interpretation, cannot but be required, it must be more wisely taken as offering its contibution to the appropriate statistical evaluation of desired behaviors, than as necessarily indicating specifically applicable design parameters irrespective of the global experience digested until that moment.

However, I shall discuss below the relevance of single layers in affecting differential settlements and cracking at the top of the dam section. Therefore we should distinguish between individual test points and the averages extracted from individual tests (based on dispersions of test values within a specific lift). The aver-

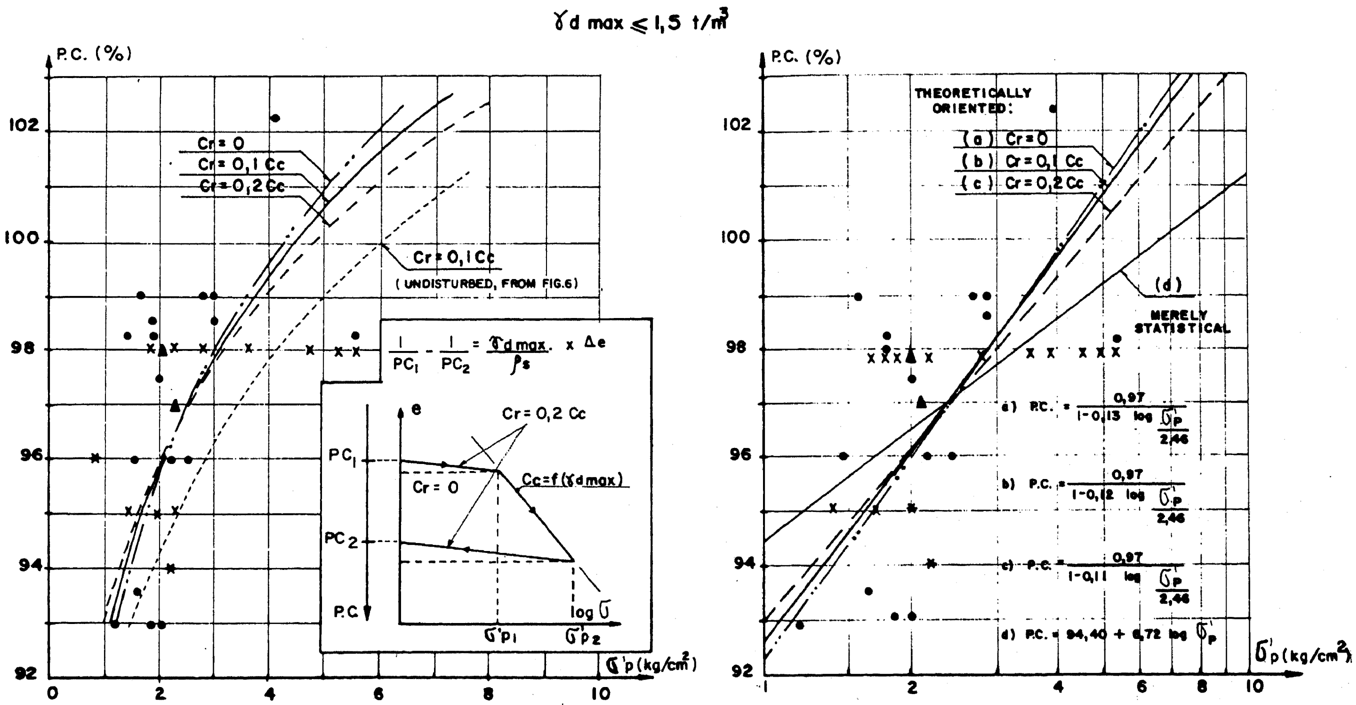


Fig. 5 - Theoretically Oriented Statistical Regressions for Compaction σ'p, Molded Specimens

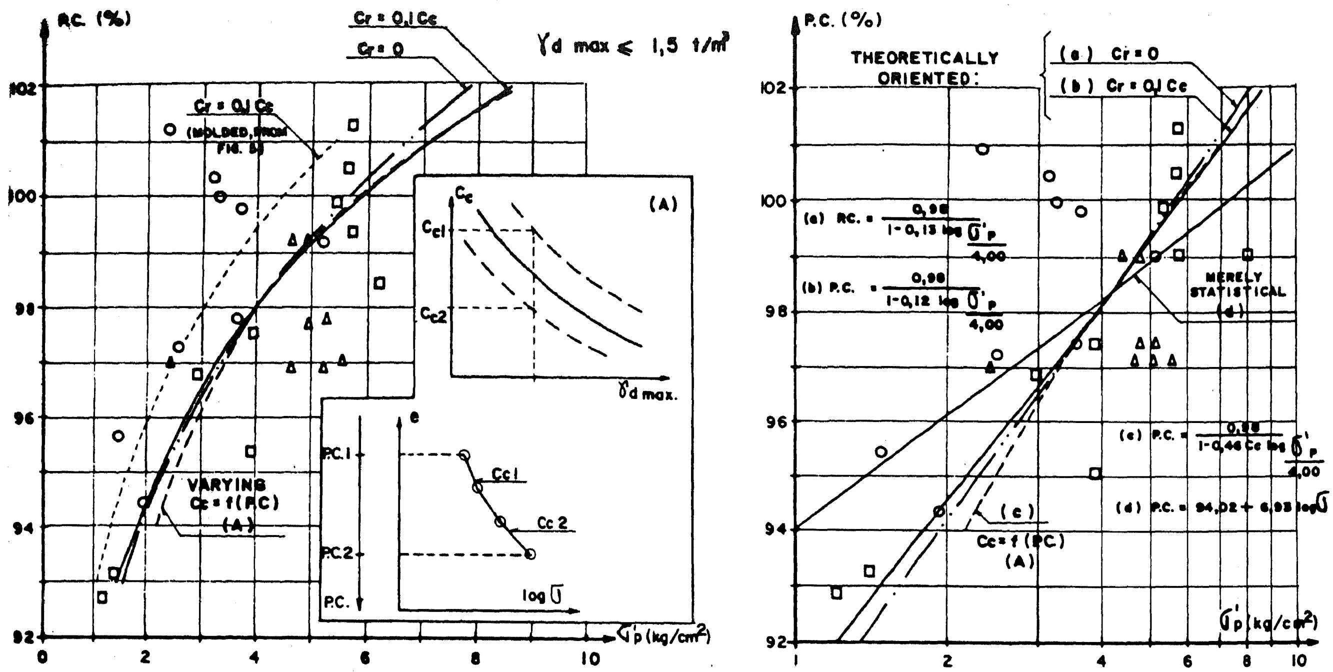


Fig. 6 - Theoretically oriented regressions on undisturbed block specimens, comparison with Fig. 5

age behavior of a given lift definable from a limited number of tests from that specific lift would present less dispersion than the individual tests themselves, while maintaining a broader dispersion band than obtains from many tests on many similar lifts. The use of different averages and confidence bands on them must be judicious.

The mental model that associates PC% with $\sigma'p$ imposes a linear statistical regression between PC and log $\sigma'p$ as summarily deduced in Fig. 5. Thus the directly available data have been reanalysed under this premisse, with separate treatments for the specimens compacted in molds in the laboratory (Fig. 5) and those trimmed from undisturbed blocks from the compacted fill (Fig. 6). The figures are prepared for convenient comparison, both in semilog and in arithmetic plots. It can be seen that laboratory compacted specimens (used in design phases) tend to indicate higher compressibilities than would be indicated by the oedometer tests on undisturbed block specimens (but even the latter still overestimate prototype settlements).

Two possible refinements were considered for investigation of significance and/or sensitivity. Firstly the idealized recompression curve (adopted straight line) was taken with some inclinations, e.g. $Cr = 0.1\ Cc$ and $0.2\ C_c$ rather than merely horizontal $Cr = 0$. It can be seen that the influence on the regressions was minimal (Fig. 5,6). Secondly, a hypothesis was adopted that as the PC% increased the C_c of the soil might reasonably decrease somewhat due to the dense structure. For a trial the decrease in C_c was taken as if it had been due to the increasing γd max (associated with sandier material). This hypothesis of varying C_c was checked only for the undisturbed specimen data (Fig. 6) and has a also proved unnecessary because relevance would be of second order.

To cover the range of materials of more common use the same types of analyses were extended to cover the moderately sandier materials (Fig. 7, $1.5 < \gamma dmax < 1.7\ t/m^3$) and the most sandy materials (Fig. 8, $1.7 < \gamma dmax < 2.0\ t/m^3$). Because of insufficient data no regression equations were attempted, but the curves and semilog straight lines were drawn as theoretically oriented, and located appropriately by eye. It can be seen that the trends of the conclusions above established are maintained, but dispersions increase appreciably. In fact, for the sandiest materials dispersions are so great that the use of the semilog graph was imposed by visual reasons. It seems reasonable to attribute the increasing dispersions principally to sampling and testing problems.

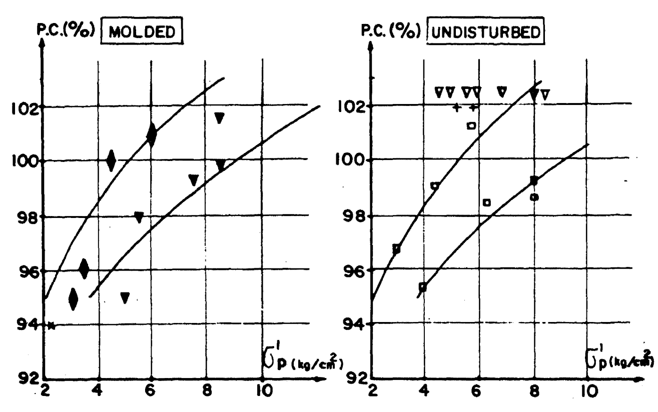


Fig. 7 - Comparative Oriented Trends for Sandier Soil (insufficient points for equations)

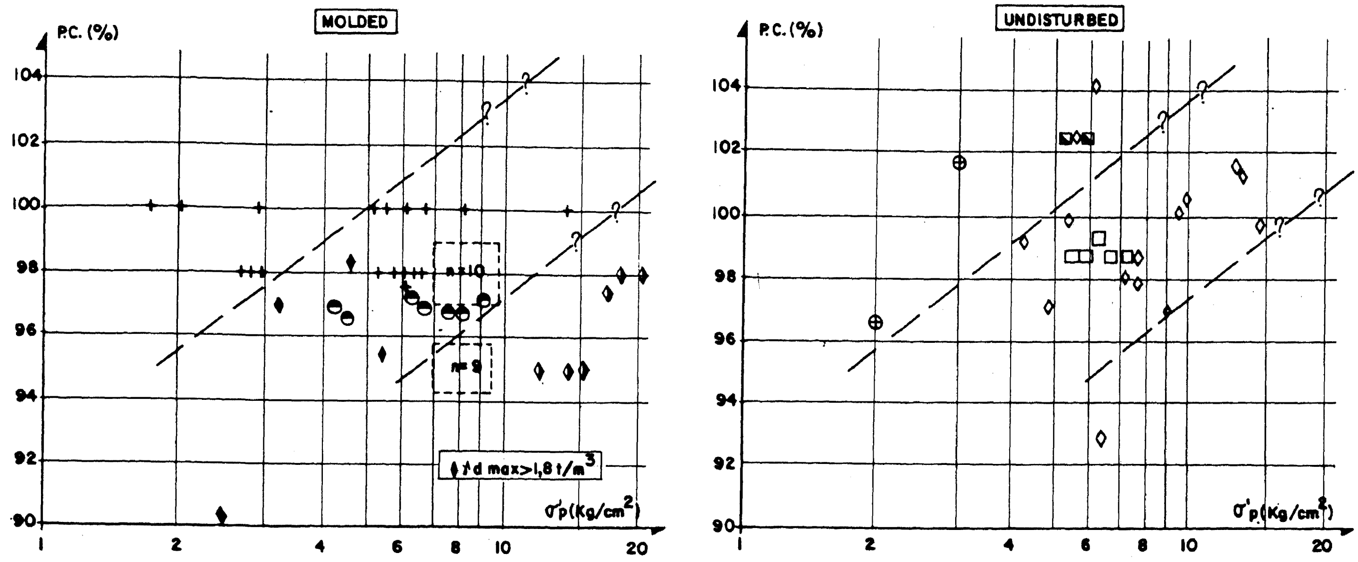


Fig. 8 - Comparative Oriented Trends for Sandiest Materials - γdmax > 1.7 t/m^3

3.2. Compression Indices C_c

Having adopted the mental model of precompressed behavior for compacted materials, and having established the statistical estimates of $\sigma'p = f(PC)$, it now behoves us to establish the statistical regression for the C_c compressibility. Our experience has favoured the use of the Standard Proctor γdmax of the various soils as a more appropriate index test for classification of the behaviors traditionally associated with more sandy vs. clayey materials. The same satisfactory type of equation published previously was reused directly, Fig. 9. Subsequently an exponential form was tried, but did not result in improvement. Finally, because of the fact that most of the very clayey materials (γdmax <1.5 t/m^3) were red residual soils of

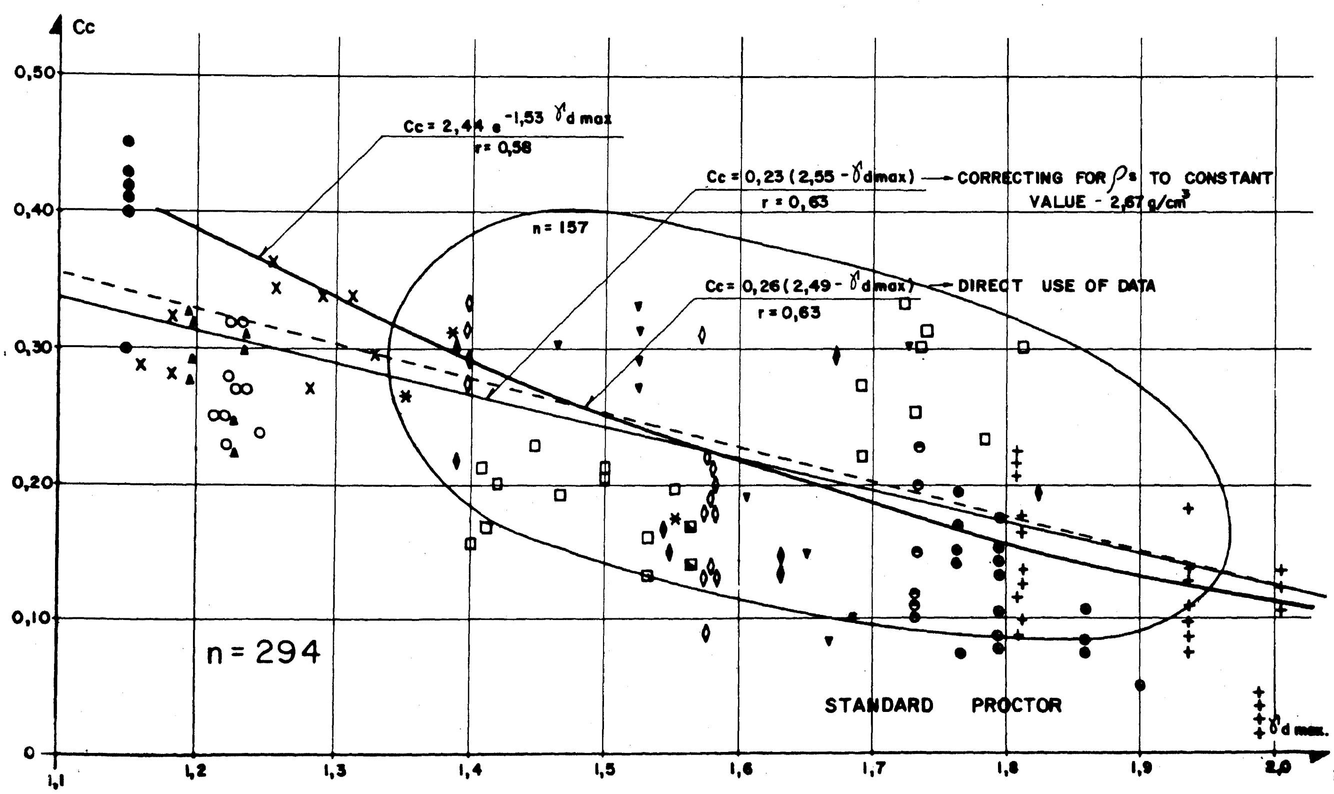


Fig. 9 - Regressions for Cc = f(γdmax) for all Oedometric Data Collected

basalts, with higher than average δ values (about 2.9 t/m^3) a correction was introduced reducing all γdmax values to a nominal value of 2.67 because of the direct association of compressibilities with porosities.

Although the final equation $C_c \approx 0.23\ (2.55 - \gamma dmax)$ changed little, both its form and its coefficients suggest themselves as most appropriate.

3.3. Overall Generalized Compressibility

If on the one hand it be contended that often (as in the present case of compressibility settlements) statistical average behaviors deduced from a broader spectrum of data might be more representative than individual specific test results, on the other hand the engineer must guard against generalizations too broad, except for the purpose of "educated guesses" for feasibility estimates. An appropriate example of this extreme is configurated by the generalized expression on volumetric compressibility proposed by Janbu (1963): Fig. 10 reproduces Janbu's suggested broad general band for all mineral granulates, and on it we have superposed our data as per Fig. 9. Although the nature of the problem is obviously similar, one can well see that when the generalizations are too sweeping, the practical interest to the engineer facing a given project is totally lost: the dispersion would permit estimates varying more than ten--fold for a given porosity. We well know from test results that although the selfsame porosiy might be reached either by preparing (mixing, depositing) a sample at a given porosity or by precompressing a looser sample to that condition, the two "structures" will yield differentiated compressibilities, and in the latter case there will be a more perceptible difference between the precompressed and the virgin compression ranges.

There is, thus, an optimum point of judicious decision between too much credence to specific test results, and too much reliance on "generalized experience".

Another direct observation is that compacted materials tend to behave (in oedometric test results), in the case of clayey materials somewhat more incompressible than anticipated, and in the case of sandier materials, much more compressible than anticipated: the latter result is known to be due to sampling, trimming and testing errors in more brittle specimens.

Two aspects yet to be discussed concern, on the one hand, the extent to which increased numbers of tests might improve definitions of design parameters, and further, the improved applicability of more sophisticated tests hitherto occasionally considered.

The first of these items will be discussed forthwith, and firstly on the basis of no more than statistical reasonings. Figs. 11 A, B present for a sample condition the curves of percent confidence bands and how they tighten as one increases the number of tests, assuming a constant statistical universe. Under the premisse that engineering parameters and decisions are based on upper and lower confidence

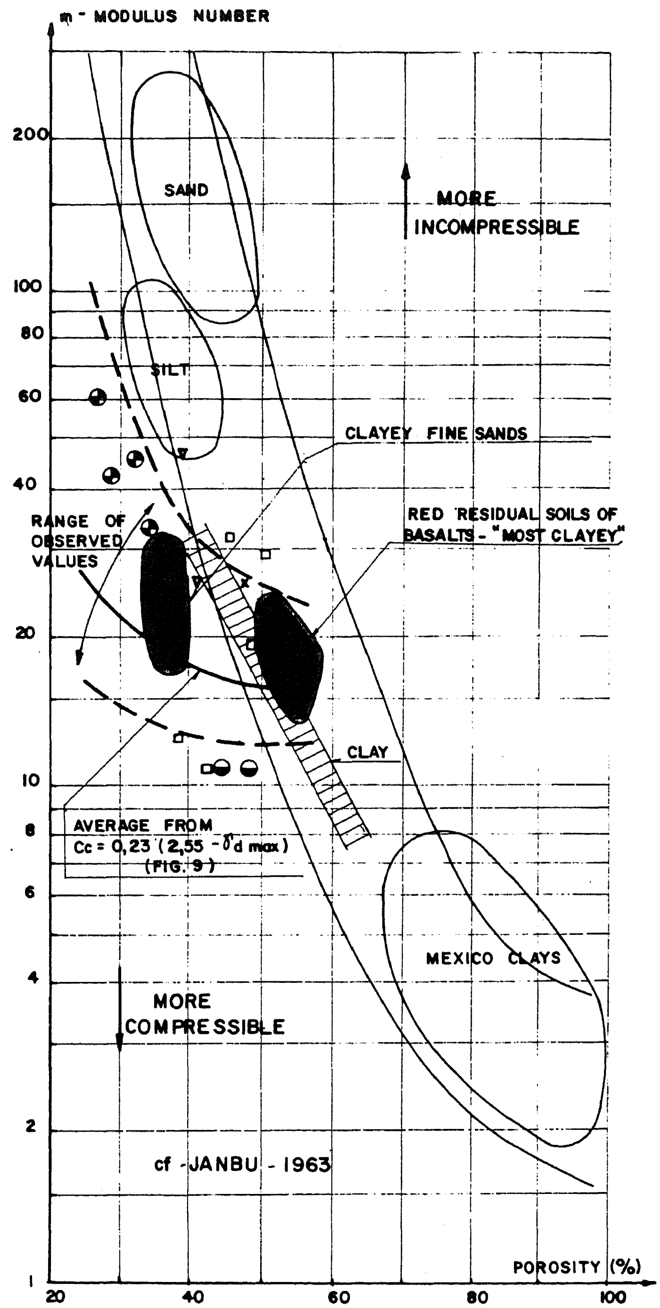


Fig. 10 - Generalized Compressibility Trends as function of porosity, all soils compared, virgin compressions, Cc conditions

bands of some sort, it is of considerable relevance to develop for any given problem the applicable rate of improvement of design bases with increased routine efforts.

It is my conviction, however, that in our cultural advances in general we do not fail to interact with the very statistical universe of the data we are progressively collecting: in other words, we should and generally do progress in the accumulation and digestion of progressive data and concomitant consequent decisions in a Bayesian manner. Fig. 11A reproduces some of

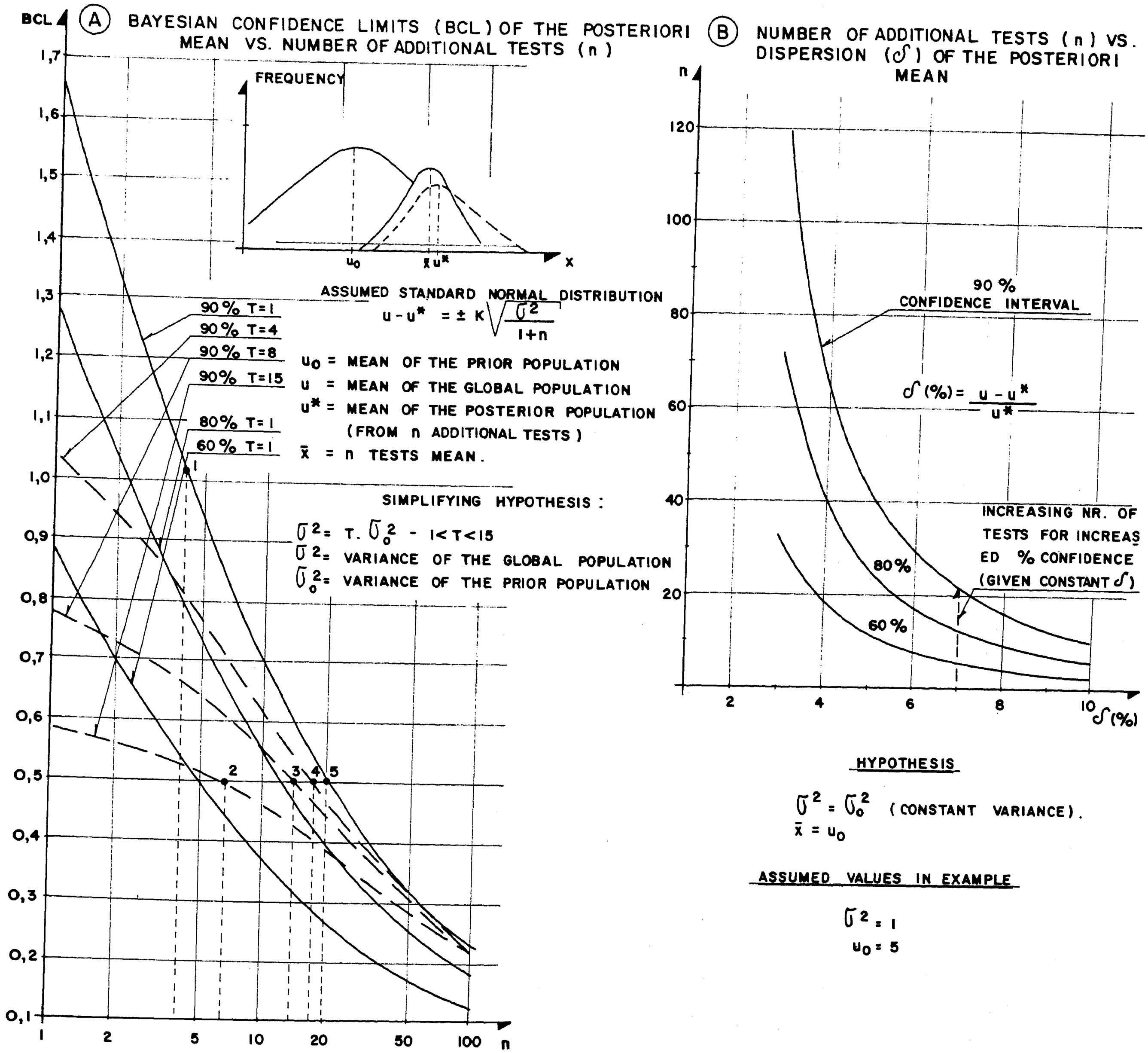


Fig. 11 - Bases for Evaluation of Gains in Decision Bands by Increased Testing

the data in a manner purporting to elucidate the advantages of such Bayesian progress from prior to posterior probabilities through improved "educated guesses" (what I have called quantified Observational Method). It can be seen that whereas in a universe of constant dispersion (T = 1.0) the gain in confidence interval IC_B from 1.04 to 0.5 would require increasing the number of tests from 4 to 20-4 = 16, if after the first step we incorporate a more educated guess for the second step (smaller dispersion, T changed from 1.0 to 8) the corresponding number of additional tests could be limited to 14 - 4 = 10 instead of the 16.

Note that the Bayesian calculations carried out are based on the routine mathematical hypothesis of symmetry, whereas we are presently searching for the desirable and necessary incorporation of computed bias, assymetrical, because it is inescapable that human (and therefore engineering) decisions are assymmetrical (greater fear of a certain result, than desire for an equivalently opposite result).

4. FIELD COMPRESSION DATA FROM SETTLEMENTS, COMPARED WITH TEST RESULTS

Figs. 12, 13, 14 and 15 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. 16 summarizes, for comparison, typical data on some settlement gages, presented both with respect to γz and with

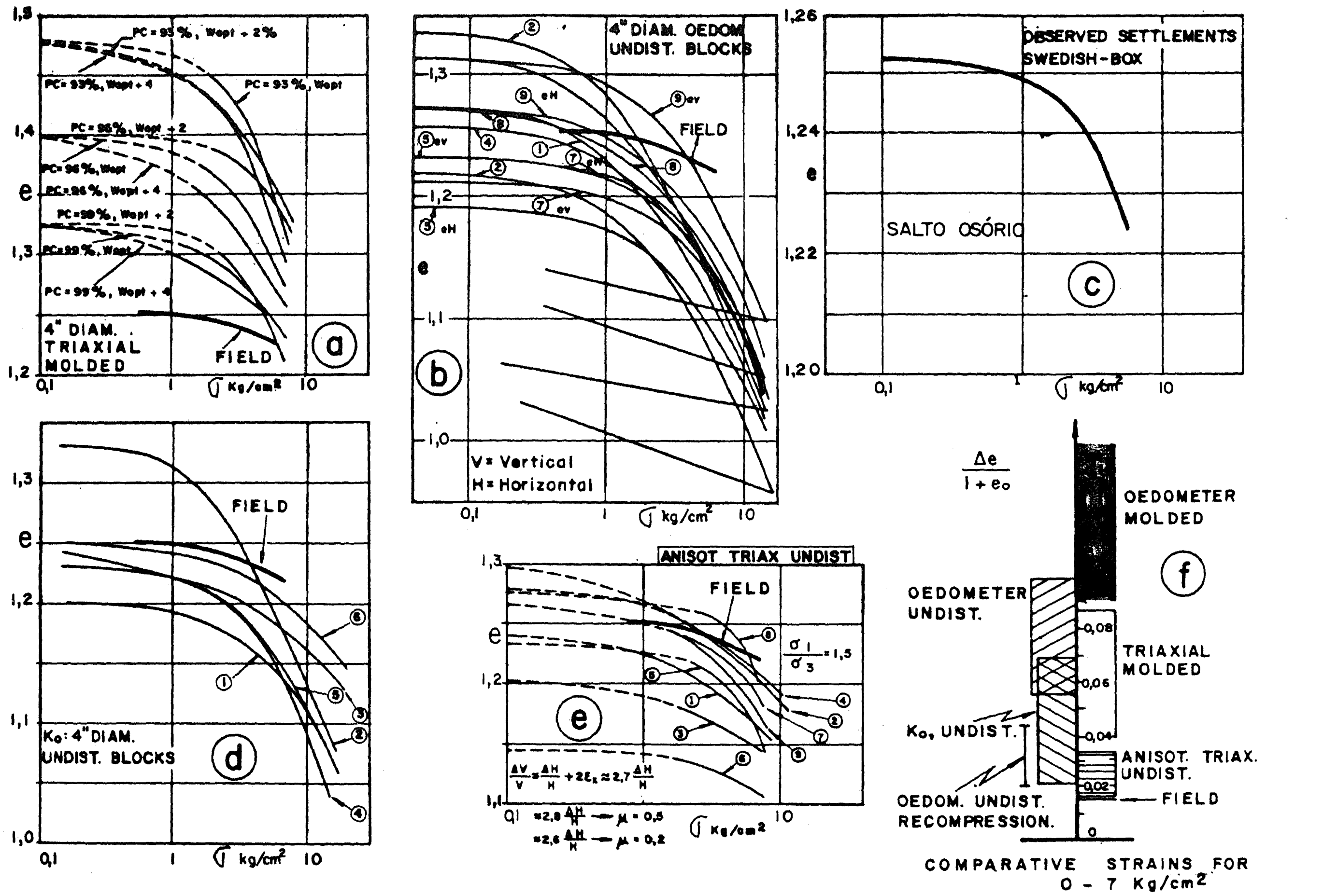


Fig. 12 - Field Compressive Strains Compared with various Lab. Tests Tried. Salto Santiago Clay Data.

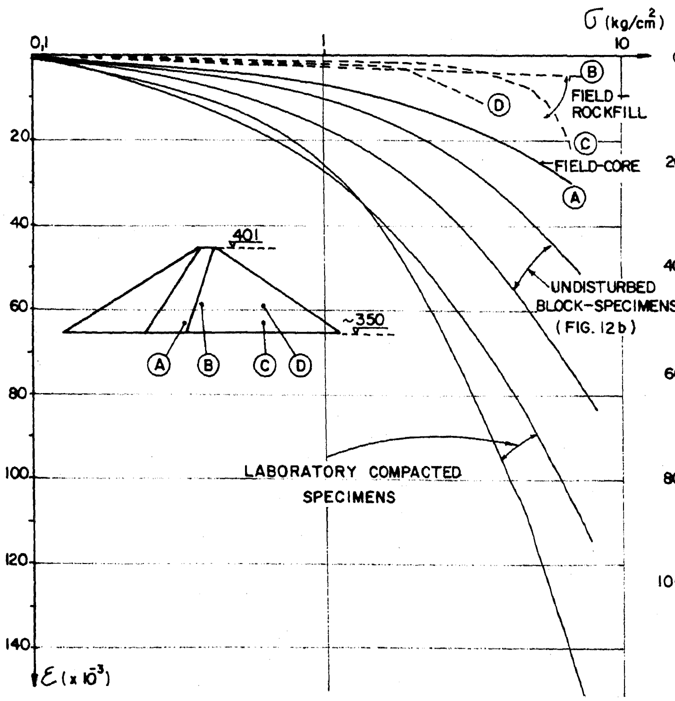


Fig. 13 - Compressions, Laboratory and Field, Salto Osorio. Indications of $\sigma'p$.

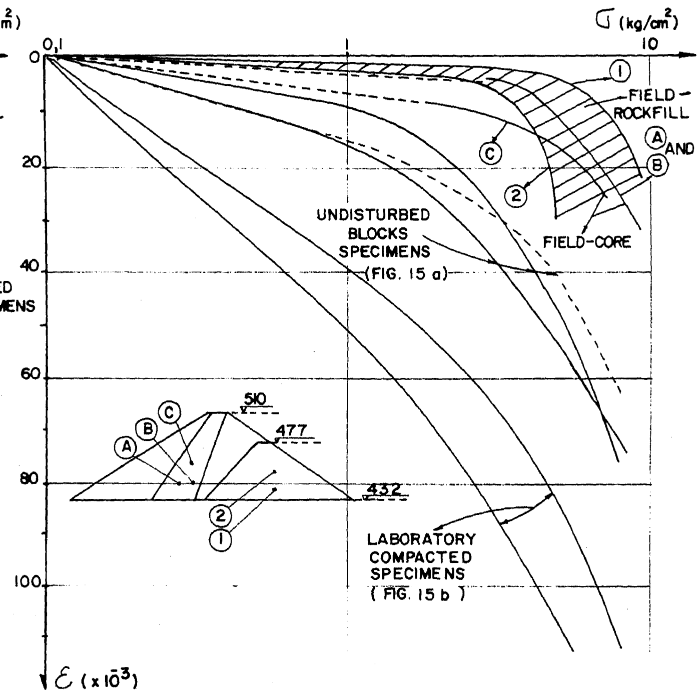


Fig. 14 - Comparative Data, Salto Santiago Dam, Lab. and Field.

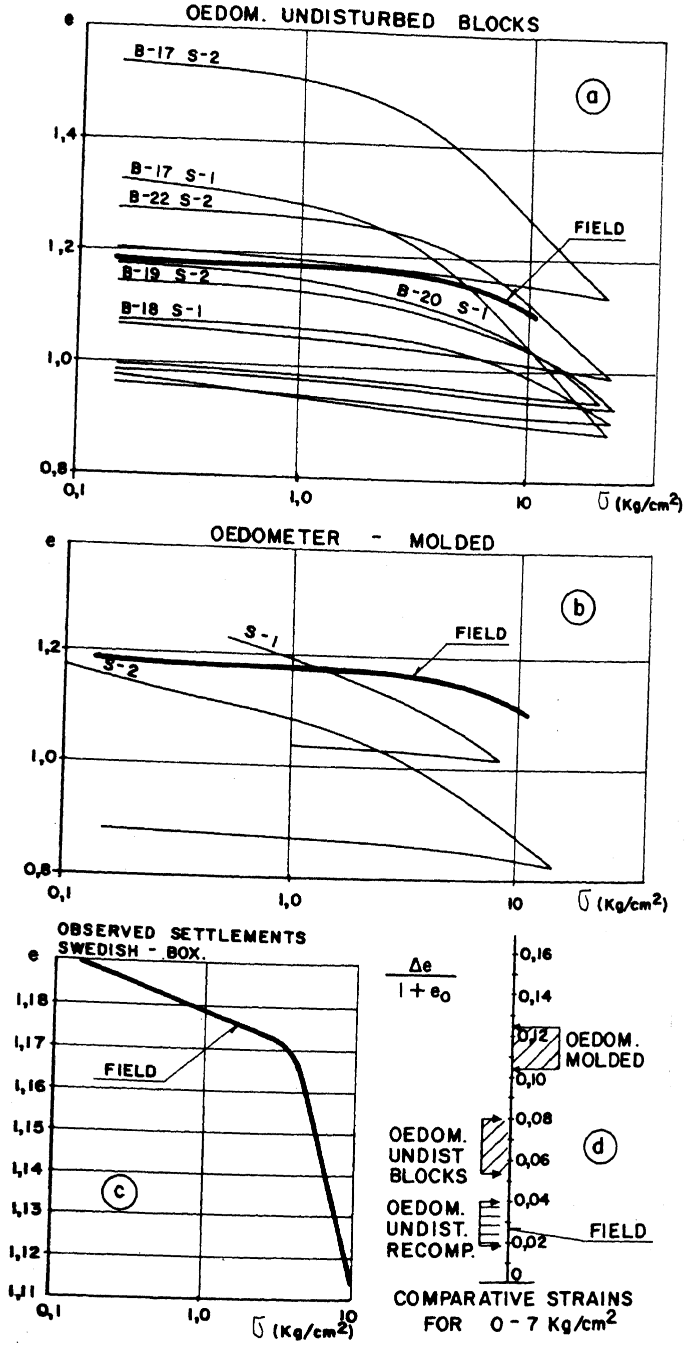


Fig. 15 - Compressibilities, Lab. and Field, Salto Santiago Clay.

respect to the nominal elastic Iγz: the principal consequence is that the log (Iγz) 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 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

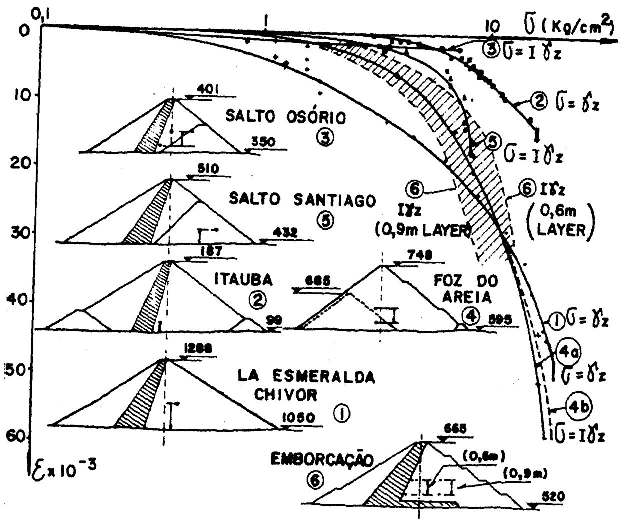


Fig. 16 - Rockfill Compressibilities. Need to include Influence I Factors for Stresses. Convenience of plotting s vs. log. σ

almost exactly the same indications: the scatter in laboratory data is apparently great, but the consistency of field observed behavior was generally 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 procedures and corresponding parameters.

Laboratory molded specimens 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.

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 σ1/σ3 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 simple 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 Sievering (1977).

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, which tallies reasonably with the observations that stress distributions between core and rockfill at the points of settlement observations were not significant.

In conclusion, it would appear that since the principal problem would derive from statistical dispersions in the tests (possibly because of small specimens etc...) and since irrespective of test-type sophistication the factors of adjustment laboratory-to-prototype are of major reductions, most often it will prove preferable to employ the more routine (oedometer) test and greater numbers of tests, for more rapid and economical determinations of the applicable adjustments in individual cases. It is irrefutable that we gain much by pursuing investigations as to which laboratory test (oedometric by diverse applicable procedures, or triaxials of various stress-strain-time trajectories) might best represent a clayey core behavior. In such a case, however, the subject falls into the areas of 1) the systematic error between laboratory soil element behavior and corresponding field soil element 2) the above-mentioned calculations of the numbers of tests (each with its dispersions, which unfortunately can often be greater, the greater the sophistication) necessary to achieve average parameters within the desired tolerances.

5. NECESSARY IMPROVEMENTS EMPLOYING DATA FROM CASE HISTORIES

It is recognized by the practising professionals that the principal source of information should derive from field data from case histories. Especially in the case of rockfill it is emphasized that "characteristics cannot be determined beforehand by laboratory tests" (Soydemir and Bjaernsli, 1979). However, it is very important to recognize that field compressibility data have most often been computed, plotted, interpreted, and published under oversimplified assumptions. At the other end finite element analyses have been employed, yielding valuable conceptual indications but often leading to undesirable conclusions when tied do back-analyses of questionably derived case-history data.

It is the intent herein to suggest to the practising professional a middle course that may be used to digest statistically the records of existing dams within a moderately plausible physical and computational model, thereupon furnishing for each new project the means not merely to reach more probable design estimates, but also, principally, to adjust such estimates through a "quantifiable Observational Method" right from the start of construction through appropriate inspection testing and behavior monitoring.

The important point to emphasize is that one cannot backanalyse published data from past projects without inquiring what data were obtained and how, and without readjusting such data to the new mental model. And that is why, once again, the use of the most modern sophisticated solution, although theoretically much better, may find itself unable to establish the link with the "experience" to be extracted from the past; whereupon at each stage of progress preference may have to fall upon a moderately theorizable improved mental model, applicable to greater numbers of case-histories, reasonably adjustable.

At the extreme of the empirical use of as many cases as possible one might cite such proposals as that of Soydemir and Bjaernsli, 1979: all displacements were related merely to H, and only post-construction data were analysed. Thus there is no mental model tying during-construction decisions and observations with future consequences, which is frustrating to the very essence of engineering. Crest movements (settlements and horizontal displacements) and displacements normal to the upstream deck were considered, for reservoir filling, and subsequent: these are indeed the points of consequence, but the engineer faces important decisions that cannot be tied passively merely to H and the categories of compacted vs. dumped rockfills.

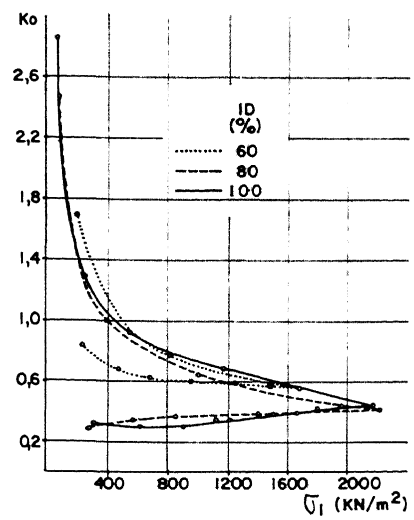
At the other extreme there have been individualized back-analyses by finite element programs, adjustable with evident success. There has been satisfactory correction of test results from oedometric and triaxial conditions to plane strain conditions for the dam through inclusion of the ν corrections; but there has been insufficient recognition of the influence of ν on field observations, partly because of inertia in adjusting assumptions comprehensibly applied in earlier settlement analyses of earth dams. Moreover, one might question if often the data to which the finite element analyses were adapted did not incorporate an incompatible simplification such as was the routine assumption of field vertical stress $\sigma_v = \gamma z$ (Boughton, 1970, on Lower River Bear Dam nº 1).

The routine assumption used to be 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 vertical comprising the point (and "compressible column") under consideration for each settlement gauge. If slopes are flat and the dam construction does not incorporate varied phasing within the section (longitudinal and transverse), such an assumption may not become patently unacceptable. In fact however, as the fill rises, a given midpoint between vertically positioned settlement gages within the dam body 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 the wrong earlier practice of interpreting and plotting construction settlement data is that at some moment when the fill height has reached the outer slope above the point, the presumed $\Delta\sigma$ is considered to have ceased, and the continuing settlement has been presumed indicative of consolidation (in clayey materials, wet, high dams) or as "secondary compressions" and "creep" in rockfills. There must be conditions in which creep occurs in more crushable rocks, in cases of more intense wet-dry and frost-defrost cycles, and so on: but in our dams herein discussed such situations could be set aside as negligible; and it is better to analyse the simpler case of instantaneous deformations.

In short, the proposal herein is to include the interference of: a) the Influence factor I (presently for homogeneous elastic bodies); b) the explicit effect of the Poisson's ratio ν so as to permit adjusting for its importance and important variations; c) recognition of the variation of compressibility with σ, and this preferably through the mental model of compaction precompression $\sigma'p$ and semilog straight line in the "virgin compression line".

Fig. 16 presents some illustrative data along such lines. Fig. 17 reproduces data from recent research testing on modelled rockfill (Veiga Pinto, 1983), which patently reinforce the indication of higher $\sigma'p$ for higher relative densities, even without perceptible crushing; obviously the effect should be greater through crushing of contacts during compaction. Finally



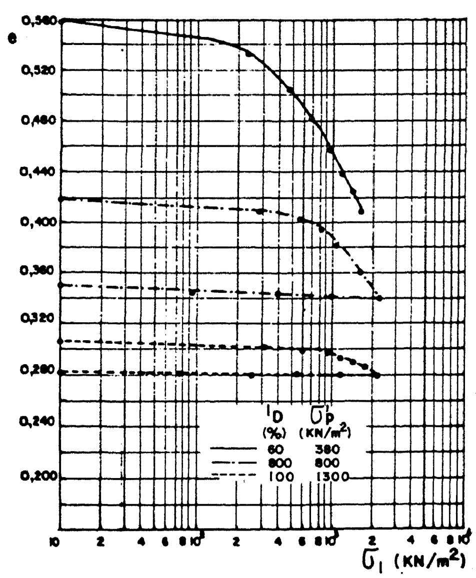


Fig. 17 - Lab. Confirmation, on Model Rockfill, Compaction Influence on $\sigma'p$ (Veiga Pinto, 1983)

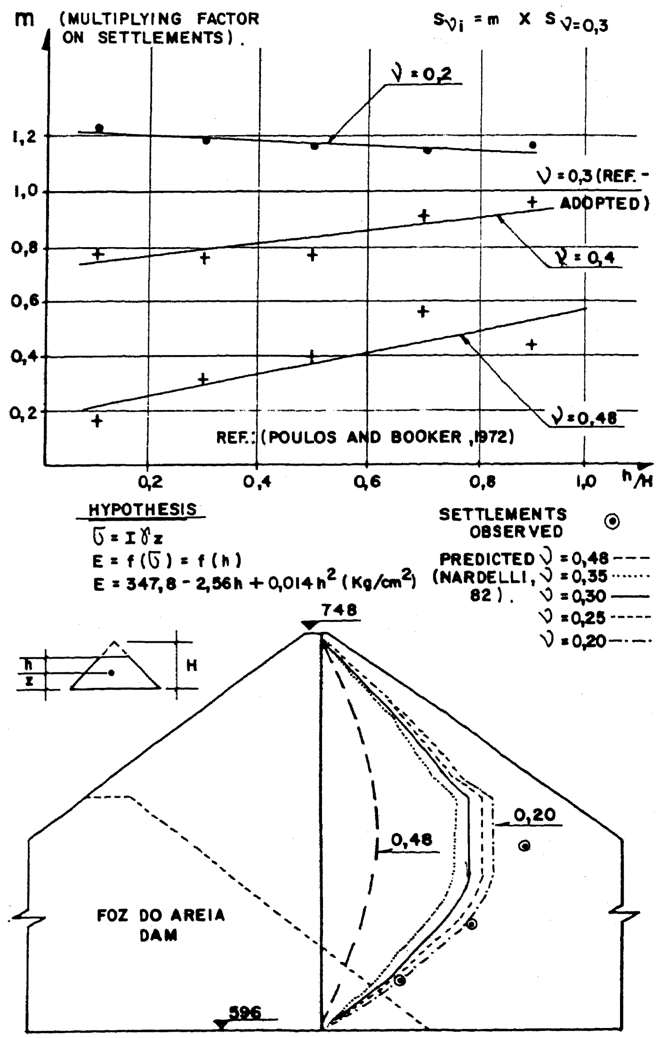


Fig. 18 - Importance of ν Values in Interpreting Self-weight Construction Settlements

Fig. 18 reproduces some recent indications (Nardelli, 1982) on the great importance of ν on self-weight settlements. The upper graph furnishes indications for preliminary estimates, in adjusting observed settlements to different ν values. As seen in the lower graph most of the surprising data on very low E values obtained in the Foz do Areia Dam might be associated with ν values, which, incidentally would affect construction settlements and upstream deck movements in a totally differentiated manner.

6. OBSERVATIONS FROM EMBANKMENT DAMS COMPARED WITH PREDICTABLE SETTLEMENTS BASED ON REGRESSIONS

The projects listed in Table I were used for reevaluation of the predictable construction period settlements compared with the prototype observations. In all cases settlements were essentially instantaneous with incremental loading. For the reevaluations the average field inspection data on $\gamma dmax$ and PC% were used in accordance with the procedures herein recommended. Fig. 19a summarizes the results showing that estimates always tend to be pessimistic, often up to 3 times higher than observed, and this principally in the higher dams and more clayey materials, wherein the fears are greater and decisions of much greater consequence and responsibility. Fig. 19b summarizes comparative data on three dams of almost geometrically similar sections, employing the very clayey material of red porous basalt clays (SO, SS and IT Dams). Finally, Fig. 19c summarizes the corresponding data for the EMB Dam, in which the intuitive precaution was taken of requiring higher PC% in a lower stretch, presuming to improve conditions in the uppermost stretch.

These data, based on the hypothesized ν=0.3, and especially the comparisons in Fig. 19b, lead to the impression that the estimated $\sigma'p$ precompressions based on oedometer tests, undisturbed

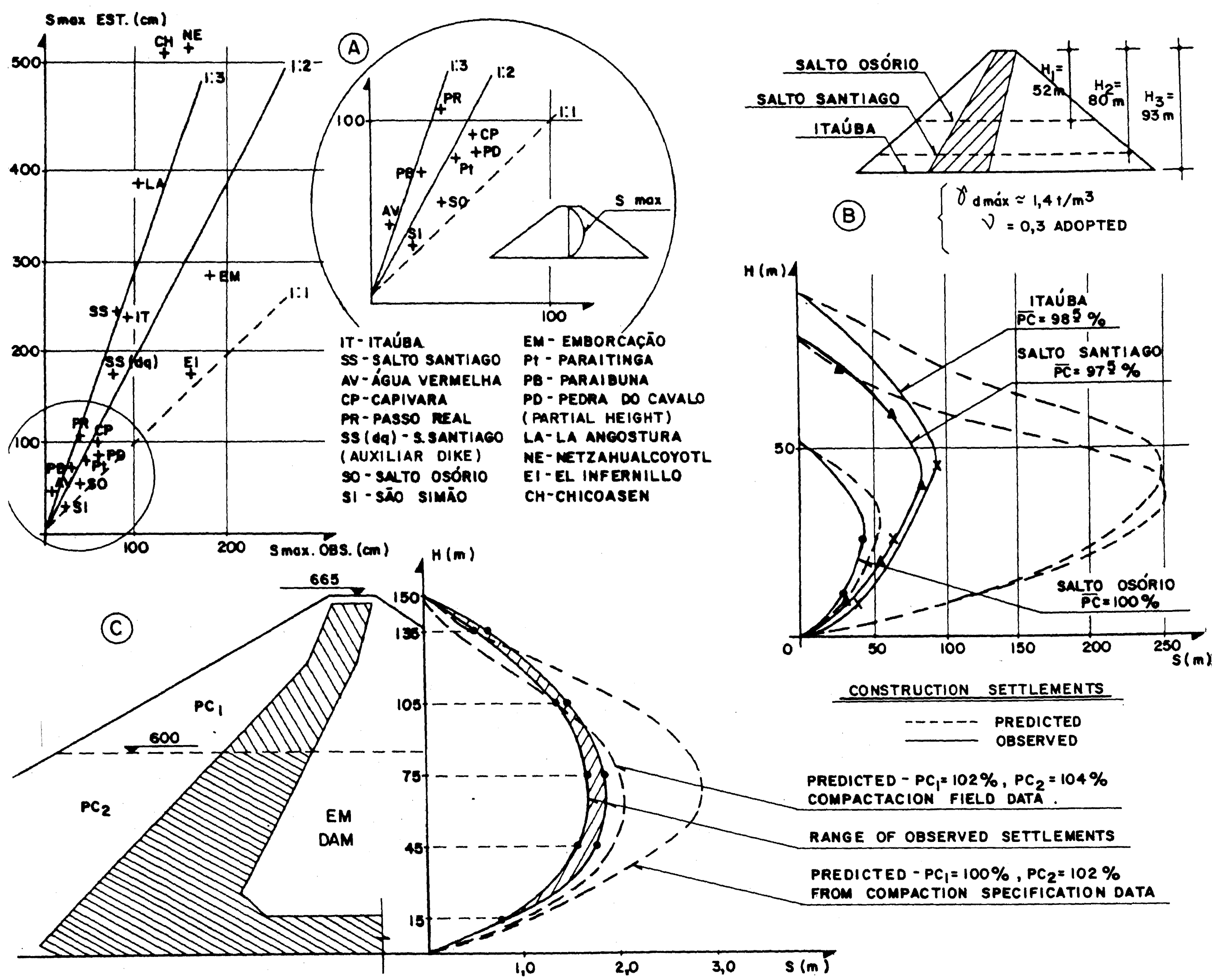


Fig. 19 - Sample Cases: Predictable vs Observed Max Settlements; Predictions Based on Present Suggestions

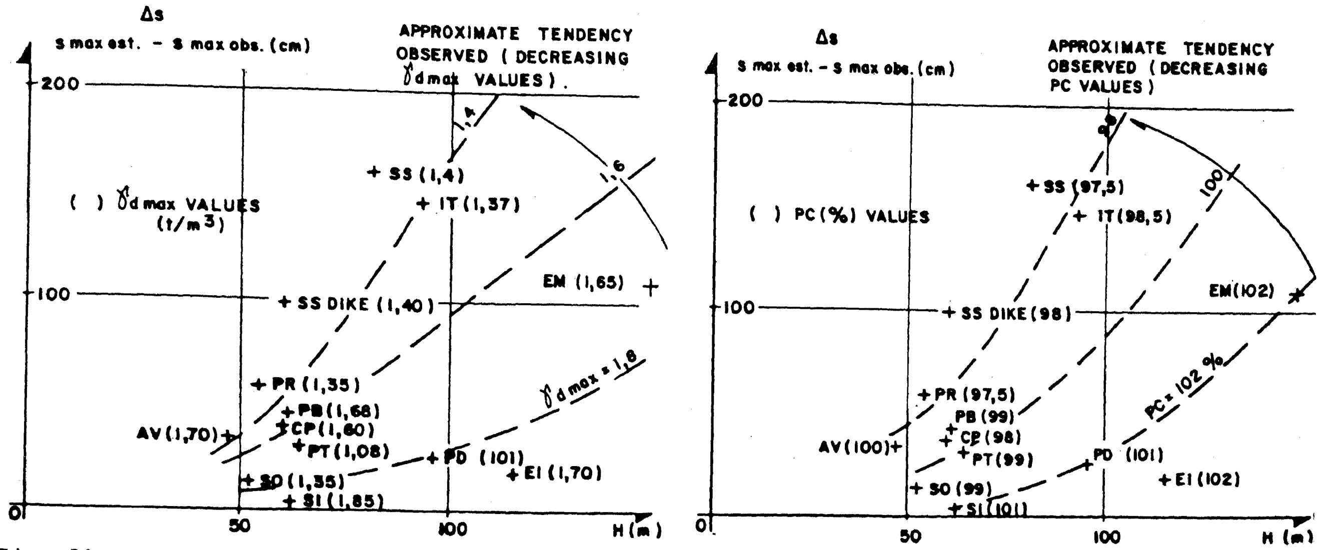


Fig. 20 - Bases for Estimation of Corrections, to arrive at smax observed from smax estimated.

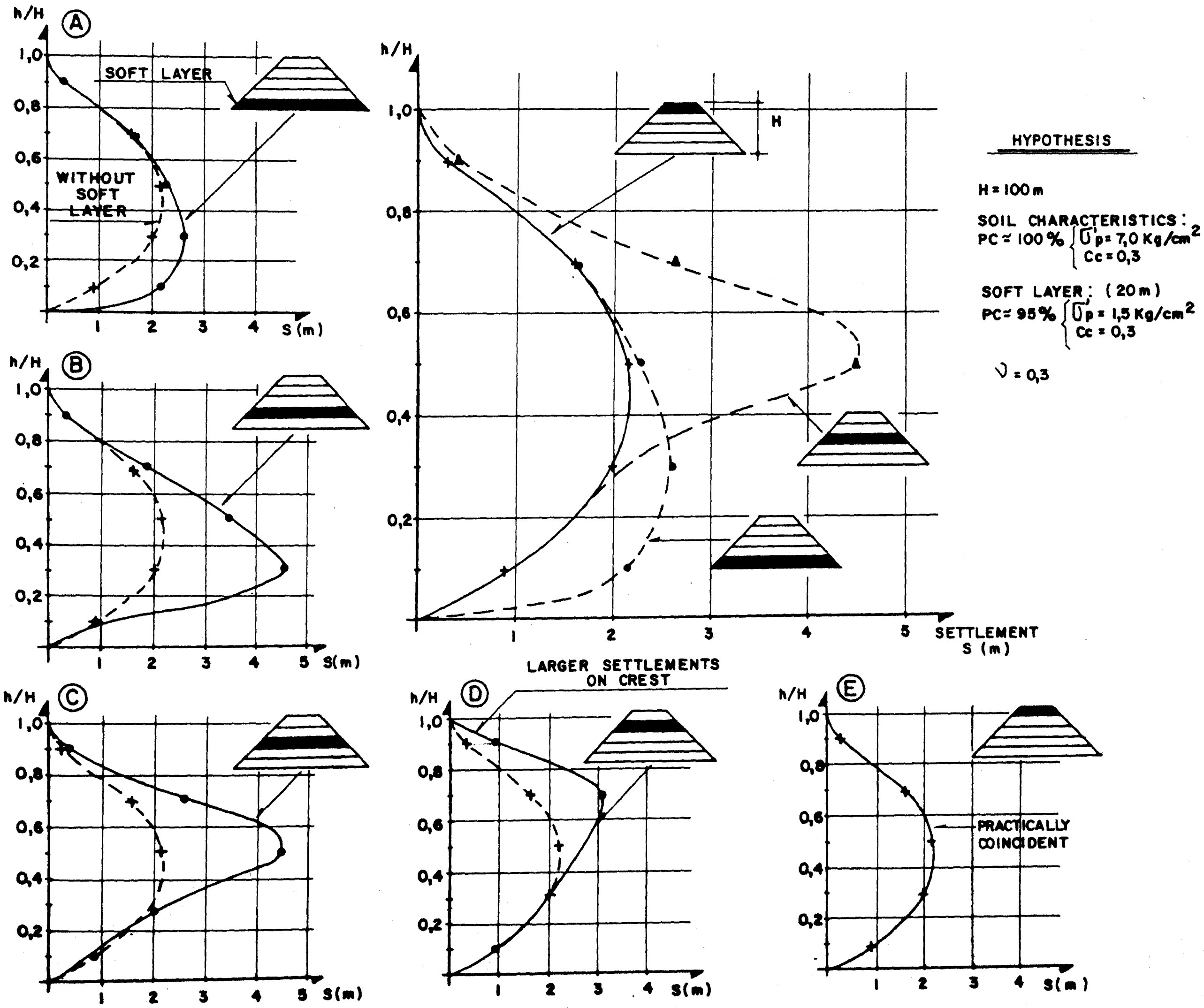


Fig. 21 - Analysis of Position of Greatest Consequence of a "Softer Layer".

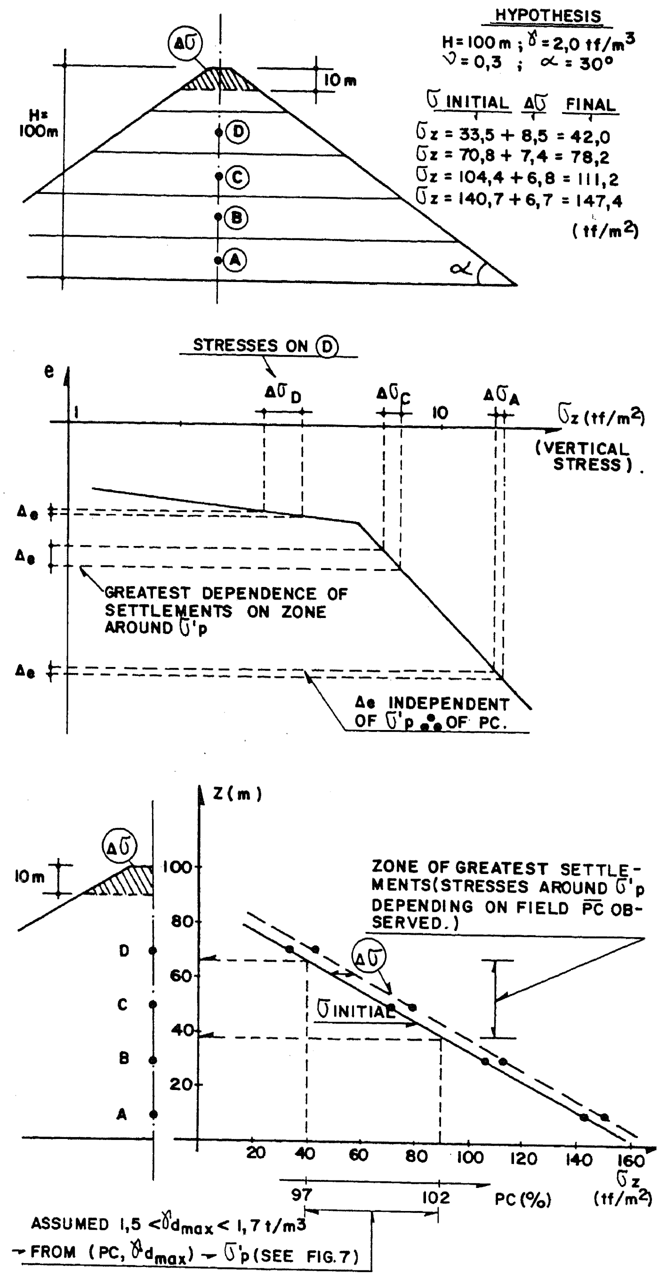


Fig. 22 - Idealized Deduction of Stretch Wherein Compaction PC Reflects Most on Top 10m

blocks, and PC% data, do not reflect adequately the presumably higher internal $\sigma'p$ prestress conditions that prevail in the dams, partly retained by the K'_o lateral compressions, and partly retained by suctions. Instrumentation and monitoring efforts will have to dedicate much more attention to problems of in situ residual stresses and transverse strains.

At present there seems to be no way to reach a better mental model, generally applicable via estimation of in situ residual stresses as functions of PC% and $\gamma dmax$. Fig. 20 has been prepared to indicate the possible trends of the interference of the two basic parameters adopted as most significant in the present mental model. Obviously the errors (and necessary corrections) increase with increasing H (and increasing predictable settlements).

Intuitions have sometimes led to requiring heavier compaction at the base of high cores, and have tended to relax such requirements in one or two steps up to the top. However, if we look at the typical internal settlement diagram (reaching a maximum at about mid-height) and if we emphasize that the only settlements (and differential settlements) that matter (to the top of the core) are those suffered by the top, we conclude that the first intuition needs immediate revision: a more compressible layer matters most where settlements are greatest, close to mid-height (Fig. 21).

In fact, by use of the mental model proposed, if we assume that attention concentrates on the top 10m (rarely could tension cracks descend below) it is possible to "theorize" on what would be for each elevation the PC values that should be respected as important (Fig. 22). At great depths both the initial σv (before adding the final 10m of dam) and the $\Delta\sigma_V$ ($I\sigma z$ corresponding to the additional final height) lie in the Cc straight line and the compression should be approximately independent of PC. Also at the uppermost elevations, both initial σv and $\sigma v + \Delta\sigma v$ would lie in the precompressed range for a given PC, and the importance of the incremental compression would be secondary. It is, therefore, in the intermediate range of elevations that specifications and inspection have to be of greater consequence to the top.

There is at present sufficiently wide recognition that field moduli E (computed with I values but assuming constant ν) present a steady variation with σ. The data extracted from the two major dams are plotted in Fig. 23. The somewhat surprising indication has been that compacted clayey fill did not prove disparagingly compressible compared to the soundest clean rockfills. Of course, the I values need to be corrected because of the not-constant E: computer solutions changing E by steps would be the typical modern approach. However, one can easily reason that the stiffer outer "arch" should decrease stresses in the center and increase them outward. Without too much added effort one can use the homogeneous elastic body Influence values, by superpositions, to revise the erstwhile estimated stresses. Fig. 24 shows how this was done assuming but two zones and a ratio of moduli of 2. The fact is that inevitably the true curve of E vs σ drops more rapidly.

7. TRANSITION FILTER-DRAINAGE MATERIALS.

It has been repeatedly published, and is by now well recognized, that the most incompressible materials in an earth-rock dam section are those of intermediate grainsize, the typical "transition strips" used for filter-drainage. Moreover, the appreciably different behavior of rounded gravels vs. angular crushed rock aggregates has been emphasized, as shown in Fig. 25.

One observes, therefore, that problems of hang-up due to differentiated settlements tend to

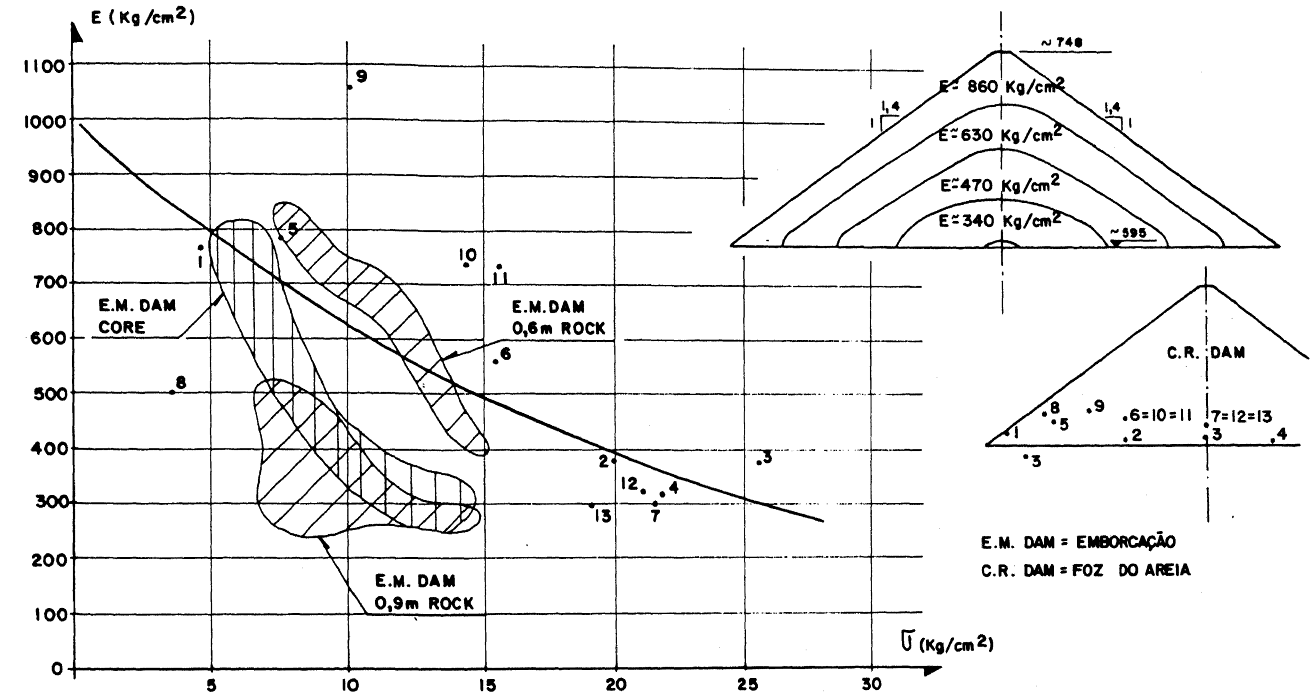


Fig. 23 - Field Moduli. Varying E with σ. Modest Differentiation between Core and Rockfill

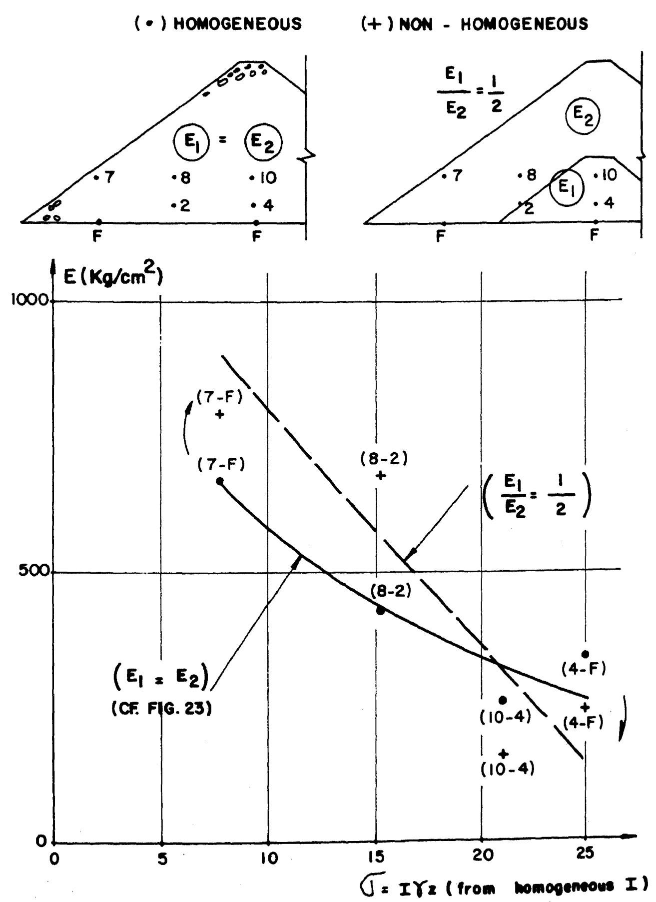


Fig. 24 - Simple Correction for Varying E. Example Assuming 2 Zones of Constant E

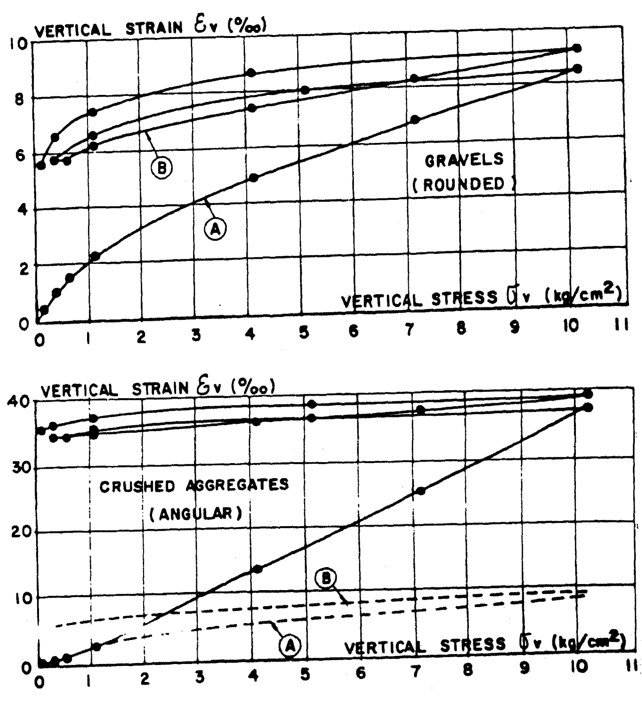


Fig. 25 - Comparative Stress-strain Behaviors of Transition Gravels vs. Crushed Rock (Kjellman & Jakobson 1955).

depend very much on the transition sections. Needless to say conditions are worsened if the design requires wider transitions, broader well-graded grainsizes, and compaction: such requirements, that would appear to add to the safety of the dam, can frequently be concluded to be exactly the opposite.

8. OBSERVED BEHAVIOR OF THE MAJOR EARTH-ROCK DAM

Presumably the crucial attentions are directed at predicting and avoiding deleterious US-DS cracking across the core. The dam in case gave the best of indications of predictable satisfactory design and monitored behavior to the end. However, shortly after the crest was (rapidly) paved, while reservoir filling was occurring (at an unexpected rate corresponding to 1:100 yr recurrence flows) two longitudinal cracks developed along the US and DS transitions in a remarkably homogeneous manner along nearly 2/3 of the length of the crest. A posteriori analyses would suggest that the upper 20m of the dam rose very rapidly, with compacted clay fill stiffer than desired; and the hurried finish of crest grading and paving served to make the cracking more salient. In short, the magnitudes of deferred movements that will affect the upper few meters are important. Moreover, reservoir filling did cause incremental rockfill settlements even in points on the downstream slope (although in modest amounts), and on hindsight one wonders how much such movements would have been attenuated if the rock-fill had been watered during compaction.

In a practical sense the case merely repeated the oft-quoted platitudes: the tension cracks (up to 10 cm wide) went down only up to about 3-4m and transformed into shear cracks (some of them into the core); there was, of course, no consequence to the dam from such longitudinal cracks, which, moreover, partly moved back, and mostly were treated by appropriate filling. It was remarkable that not a hairline fissure developed in the US-DS direction across the crest pavement, which indirectly served as an excellent visual index for comparative monitoring: the specific differential settlements longitudinally ranged in the order of 1:600 to 1:800, which are strains too minute for design estimates but have been discussed with regard to choices of appropriate materials for avoiding crest transverse cracking. What would have been the important transverse behavior if total settlements had been significantly bigger as predictable, and especially so in the final upper stretch ? I submit that it is much too specious to discuss different tensile strains to fissuring as achievable by materials of different plasticity indices and compaction specifications. For weakest link tensile conditions the reliance on meticulous homogeneity is far-fetched and too dangerous.

One practical design decision (for a subsequent project) arose both from the desire to minimize differentiated narrow strips of placement and compaction near the top, and from the theoretical desire to employ as homogeneous a section as possible in the upper 20 meters. Fig. 26 illustrates one such case wherein the only dominant feature to be retained is the chimney filter; the upstream rip-rap and transition zones could well be narrowed. Ideally any differential settlements of the differently supported zones should be absorbed along modestly compacted chimneys of (alluvial) sand.

The remarkable difference in behaviors evidenced with regard to non-consequential longitudinal cracking and the transverse cracking that would be critical, points to the need for closer attention to lateral stresses $\sigma'2$ or $\sigma'y$ in comparison with $\sigma'3$ or $\sigma'x$ that has dominated attention because of routine slope stability analyses. Fig. 27 schematically compares, on the basis of elastic behaviors (E, ν parameters) what lateral stresses would be required for closing a given crack, either longitudinal or transverse. Because of settlement, and assuming that longitudinally the dam's crest has to compress as it descends into the wedge between the two abutments, the transverse crack can be closed by a very small incremental $\sigma'y$ (residual stresses due to compaction were not considered, conservatively assumed equivalent in the two cases). The necessary incremental $\sigma'x$ stresses to close the longitudinal crack would, for the case in question, be of the order of 33 times higher. For both cases another rough computation that seems to bear interest is what height of added fill (and consequent incremental σ'_x or σ'_y) would cause the strain to counter the crack. The additional heights of fill required for the two cases are even more blatantly different because of different I coefficients. By inverting the reasoning such indication of additional fill reflects how deep a tensile crack can propagate.

An important inference from the case is that instrumentation needs on such dams have to be significantly reconsidered. Across the shell-to-core transitions it would be of interest to employ profilers for simultaneous vertical and horizontal displacements, but one must guard

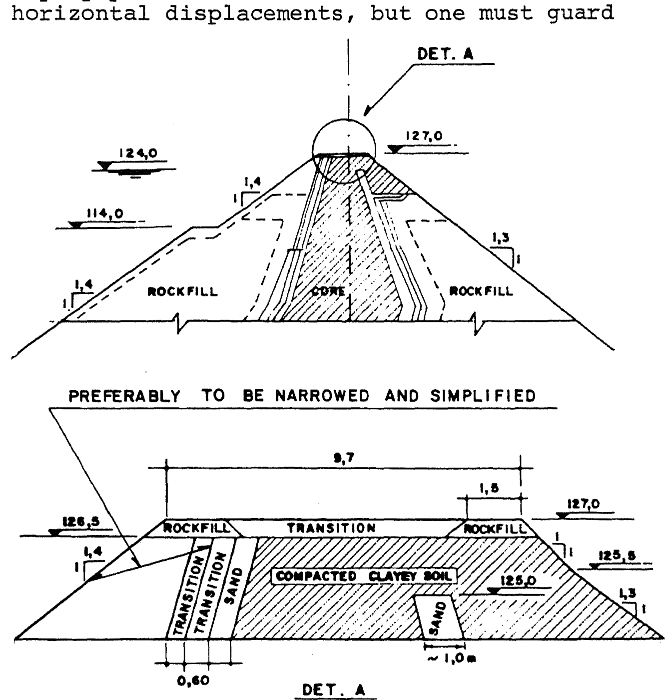


Fig. 26 - Suggested Design Revision Near Crest

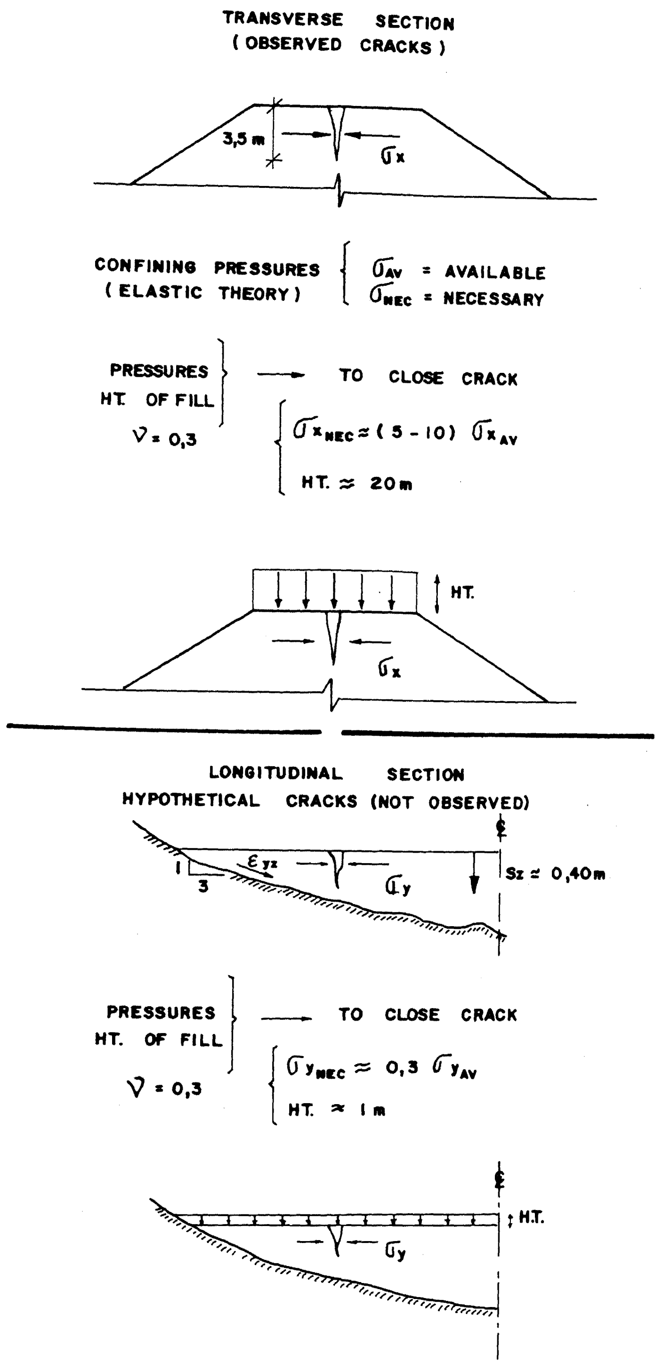


Fig. 27 - Comparison of lateral stresses required to close longitudinal vs. transverse cracks.

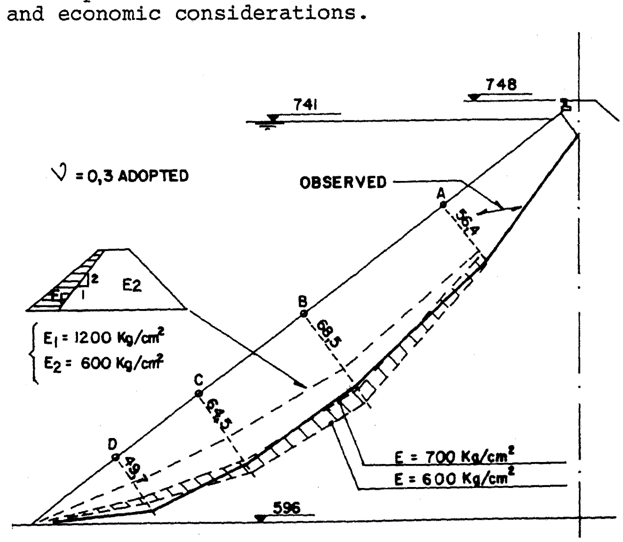
against the great danger of such instruments themselves being conducive to concentrated leaks and piping. Much more attention should be given to deferred settlements affecting the top, and to behaviors at the top.

9. OBSERVED BEHAVIOR OF CONCRETE-FACE DAM

The only concern in the case of upstream face dams lies in the displacements of the face upon reservoir filling: the need is to minimize them in order to avoid cracking and leakage. The systematic observation of construction-period deformations (hitherto almost exclusively settlements) for calculation of E moduli has only one purpose, which is the prediction of the deformations of consequence to the slab. The very low moduli computed as approximately valid towards the end of construction should have led to deep concern regarding the possible excessive opening of joints. In fact, however, the observed deformations during the first reservoir filling were only about 1/3 of the values that would be computed from the final self-weight settlement E values, Fig. 28.

The design solution for such dams has already successfully adopted the practice of employing a well-compacted finer-graded transition in the upstream zone. Doubtless this is a factor of significant benefit (as shown in Fig. 28 for an idealized case calculated by finite elements), and there might be insufficient interest or need to delve into better comprehension of the real behaviors.

I have emphasized above the great relevance of ν, and the need to monitor much more intensely the internal horizontal displacements accompanying the construction settlements. Depending on such indications one may well conclude that much of the difference (favorable) in behavior during "active" self-weight movements and "partially passive thrust" reservoir loading movements may be explainable by the very significant changes of ν for compacted rockfill under the partial stress reversal. The basic need and interest is to reach a satisfactory comprehension of the geomechanical behaviors at play. Of course, if the proportional benefits due to the finer-graded zone are better assessed, there will be conditions for optimizing the use of this special crushed-rock zone from technical and economic considerations.



USING AVAILABLE SOLUTION (REF.: POULOS & BOOKER, 73)

Fig. 28 - Concrete face displacements due to US loading.

10. CONCLUDING REMARKS

The road to knowledge ahead of us stretches ever further and wider. There is a natural tendency to stop at the oases, and possibly, to delay too long before proceeding onwards. The milestones of satisfactory engineering decisions have to be ahead of our trekking in collecting quantifiable data for digestion of the laws of the behaviors that have proven adequate. It is strongly suggested that the most profitable directions and advances require a balance between the extremes of sophisticated theoretical developments applied to single special cases, and the attempt to digest nominal behavioral information that constitutes "practical experience".

REFERENCES

Boughton, N. (1970). Elastic Analysis for Behavior of Rockfill, Journal Soil Mech. and Found. Division, ASCE, vol. 96, nº SM5, Sept. 1970.

Clough, R.W. & Woodward, R.J., (1967). Analysis of Embankment Stresses and Deformations, Journal Soil Mech. and Found. Division, ASCE, vol. 93, nº SM4, July 1967.

de Mello, V.F.B. (1977). Reflections on Design Decisions of Practical Significance to Embankment Dams, Geotechnique, vol. 27, nº 3 279-355.

de Mello, V.F.B. (1980) Comparative Behaviors of Similar Compacted Earthrock Dams in Basalt Geology in Brazil, International Symposium on Problems and Practice of Dam Engineering, Bangkok, vol. I, 61-79, Dec. 1980.

de Mello V.F.B. (1983) Design Trends on Large Rockfill Dams and Purposeful Monitoring Needs, International Conference on Instrumentation, Zurich, Sept. 1983.

Hynes, M.E. & Vanmarke, E.H. (1977) Reliability of Embankment Performance Predictions, Mechanics in Engineering, University of Waterloo Press.

Janbu, N. (1963) Soil Compressibility as Determined by Oedometer and Triaxial Tests, ECSMFE, Wiesbaden, vol. 1, 19-25.

Kjellman, W. & Jakobson, B. (1955) Some Relations between Stress and Strain in Coarse--Grained Cohesionless Materials, Royal Swedish Geoth. Inst., nº 9, Stockholm 1955.

Pinto, A.A.V. (1983) Previsão do Comportamento Estrutural de Barragens de Enrocamento, LNEC, Lisboa, PORTUGAL. Julho 1983.

Poulos H.G. & Booker, J.R. (1972) Simplified Calculation of Embankment Deformations, Univ. of Sydney, Research Report nº 192, May 1972, Sydney, AUSTRALIA.

Poulos H.G. & Booker, J.R. (1973) Embankment Deformations due to Water Loads, Research Report nº 224, Sept. 1973, Univ. of Sydney, AUSTRALIA.

Poulos H.G. & Davis, E.H. (1974) Elastic Solutions for Soil and Rock Mechanics, John Wiley & Sons, Inc., 1974, New York, USA.

Nardelli Rosi, M. (1982) Foz do Areia - Retroanálise pelo Método dos Elementos Finitos, Tese de Mestrado, Dez. 1982, PUC, Rio de Janeiro, BRAZIL.

Schultze, E. & Sievering, W. (1977), Statistical Evaluation of Settlement Observations, 9th Int. Conf. on Soil Mech. and Found. Eng., Tokyo, vol. I, 711-714.

Soydemir, C. & Kjaernsli, B. (1979) Deformation of Membrane Faced Rockfill Dams, 7th ECSMFE, Brighton, U.K., vol. 3, 281-284.

Sharma, H.D. et al. (1976). Generalisation of Sequential Non-Linear Analysis - A Study of Rockfill Dam with Joint Elements, 2nd Int.Conf. on Numerical Methods in Geomechanics, ASCE, Virginia, USA, vol. II, 662-685.

Tanaka, T. & Nakano, R. (1976) Finite Element Analysis of Miyama Rockfill Dam, 2nd Int.Conf. on Numerical Methods in Geomechanics, ASCE, Virginia, USA, vol. II, 650-661, June 1976.