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# Some lessons from unsuspected, real and fictitious problems in earth dam engineering in Brazil



PROFESSOR VICTOR F.B. DE MELLO *Professor of Soil Mechanics, University of Sao Paulo, Brazil*

The Organizing Committee's invitation to me to deliver a special lecture on Embankment Dams to this gathering does me great honour, and gives me the greatest pleasure. Indeed, I am most grateful for having been your guest at this Conference, and for having had the many and very pleasant opportunities to exchange views with you on subjects of mutual interest. Moreover, the topic suggested for my presentation is one that challenges our deepest professional enthusiasms, not merely because of its complexities and difficulties, but also because of the rewarding feeling of achievement that one derives from harnessing Nature's biggest non-expendable power for the benefit of all. I need not emphasize that in Dam Engineering, possibly more than in any other sector of Civil Engineering, one is emphatically made conscious of the fact that we only embrace a profession (such as Civil Engineering) in order better to fulfill ourselves as men and citizens, and within Civil Engineering we restrict ourselves to a specialization such as Soil Mechanics, again in order better to fulfill ourselves as fully-fledged Civil Engineers. So, as man begins to face ever more exacting problems, technical and economical, in the harnessing and use of the energy required for development, we should never forget the basic order of primordial precedence of values, Man - Engineer - Specialist, whereby our efforts have to be judged.

Thus, it is on the soil engineering aspects of dam design and construction that I shall concentrate this presentation, with the intention of conveying some of the many lessons that have been slowly learnt about the routine assumptions and doctrines of our specialization. But I am compelled to state, right from the start, that in my experience it is principally in the connection between Soil Mechanics and the overall field of Civil Engineering, and in our obligations as members of a society, that the greatest challenge and chances for creative vision beckon us and lead us forward.

Brazilian earth and earth-rock dam engineering can be said to have entered the modern era in 1948, when Karl Terzaghi was called to consult on the Sierra Slide adjacent to the Light and Power Company's Cubatão Power Plant.

Very much has been done since, much of it proving gradually that many of the early conventional recommendations by and hypotheses of Soils Engineers were too pessimistic. Construction and inspection practice has doubtless advanced very far in comparison to the early days when Terzaghi felt called upon to write an appendix to a consulting report, detailing the procedure for the determination of water contents, soil densities, and so on. And yet it would almost seem as though the respect for a visually interpreted precedent of cross-sections would stifle the acceptance of proven possibilities for streamlining the cross-section of the "homogeneously compacted clay dam".

It should be emphasized that whenever precedent is cited in dam engineering and in slope stability, closer investigation reveals that the would-be precedent is merely geometric; for instance, consider the use of geometrically similar cross-sections and constant slopes. However, almost none of the laws of behaviour at stake have any connection with the linear functions that are implicit in geometric similitude; therefore, any true reliance on and respect for precedent, duly interpreted, should ipso facto suggest a change of cross-section as the dimensions change.

The discussion must needs start with a description of the Vigario Dyke and Dam which was renamed the Terzaghi Dam in 1963. Figure 1 shows the cross-section adopted by Terzaghi. At the time there were no reasonable borings and absolutely no soil tests. The saddle dykes required no diversion, no cofferdams, and absolutely no consideration of hydrology or hydraulics because of the peculiarity of the hydro-complex, which was somewhat akin to a pump storage scheme. Moreover, there were no hydraulic structures or concrete bodies, on or against which to tie the dam. The soils were saprolites from advanced weathering of the Archaean gneiss-granites. A typical grainsize distribution curve ranged from sand to clay and had a Non-Uniformity Ratio D60/D10 higher than 100, and with what we presently recognise as a clay-potentiality represented by a Clay Percent Fraction of about 15%, a Liquid Limit of about 38% and a Plastic Limit of about 28% (see Figure 4).

The reason why in saprolites we consider the conventional soil mechanics characterization tests (for example grainsize, plasticity limits and compaction) to be indicative of clay-potentiality, and not truly of in-situ clay content, is because saprolites, especially of igneous and metamorphic rocks, behave

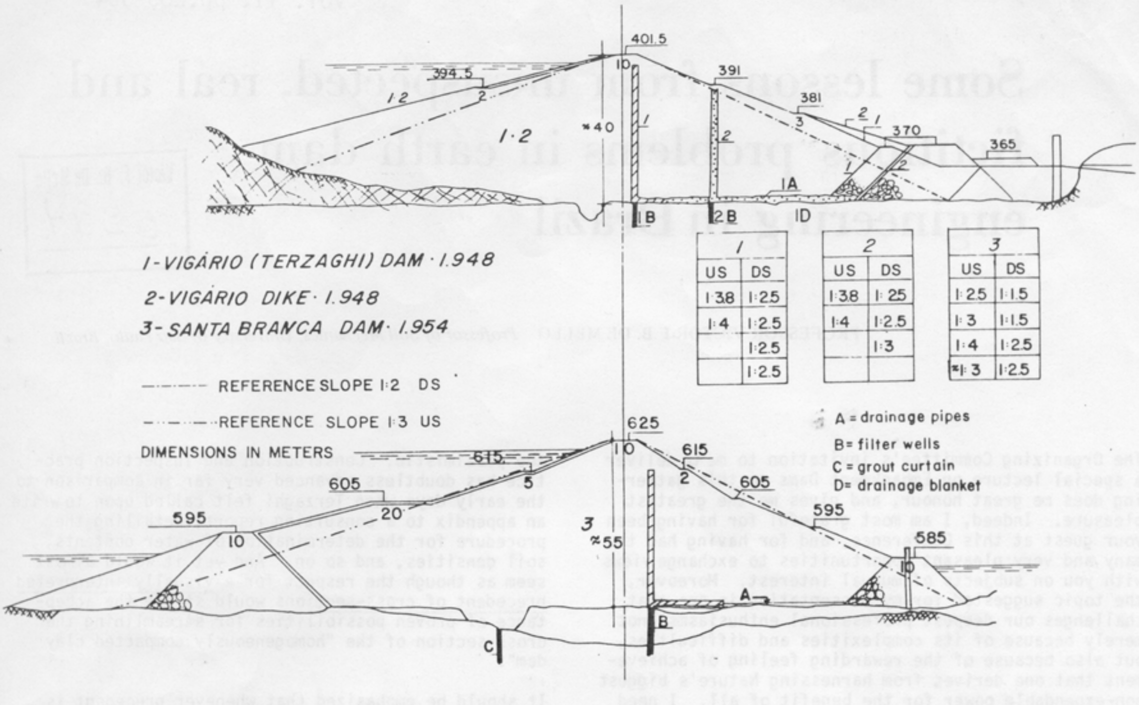


FIG.1

EARLY DAMS, GRANITO-GNEISS FOUNDATIONS AND COMPACTED RESIDUAL SOILS

in-situ as though they were conditioned by multiple-particle granules and the clay particles are not fully "plasticized" to the condition corresponding to an adequate development of the individual lyospheres. You should compare this with the paper I presented at the Third Southeast Asian Conference on Soil Engineering in Hong Kong, 1972.

The routine Proctor test at that time was run on samples which were dried and pulverized. The material was re-used for the different compaction points in order to determine the curve which gave, on the average, the following results: 1,6 tonnes/m3 maximum dry density and 20% optimum moisture content.

The materials were duly classified as clays, and accordingly there was considerable concern with respect to construction pore pressures, all the more so since the average annual rainfall was about 2,5m.

Based on the United States Bureau of Reclamation publications (Gould, etc.), it was common at the time to assume that the more clayey soils, especially the "residual soils" would be particularly susceptible to instability. To quote from these publications :

"The soils whose stability is most threatened by construction pore pressures range from plastic clays to clayey san. with gravel. They are characterized by substantial amounts of clay of at least moderate plasticity and include residual clays, fine-grained sediments ------------".

Moreover, as regards the design and construction of the dam, the inevitable implication, both because of the peculiarity of the exclusion of the fundamental hydraulic structures, termed "appurtenant structures", and because of an exceptional fear of the unknown discipline of Soil Mechanics, was that the design of the dam cross-section was conditioned by the dominating and in fact independent discipline of Soil Mechanics.

Indeed, Terzaghi's solution to these design problems may be seen as a remarkable case which exemplifies the distinction between that aspect of Engineering which comprises decision and action despite uncertainties, and which necessarily seeks to transcend the statistical universe of incertitudes and the much-too-frequent modern trend towards scientific pseudo-engineering, which is preoccupied with an ever-deeper delving into the problems faced, and towards a minuter quantification of these particular problems. This solution may be said to represent another striking example of the fact that design really implies a decision which often transcends that of Decision Theory. Good design is not yet cornered; from its position of affluence of ingenious ideas, it will move into the realm of better calculation or better estimations of the intangible risks. No matter how much we do really rely on such improvements for the iterative progress of science and engineering, we must act in this way.

This presentation is aimed at the design engineer who has a moderate soil mechanics background, and therefore its keynotes may be summarized as incorporating the following fundamental items :

1. Engineering Geology, that is Geology which is quantified and synthesized for a technological purpose. Engineering Geology is imposed on you at the site, and must be carefully taken into account in quantitative terms.

   If a Geologist declares that at a given site the joints strike unfavourably in an upstream-downstream direction and tend to open to significant depths, and therefore the site should be abandoned, as a Civil Engineer I would say :

   (a) accept the first part of the statement, as the information comes from the appropriate source,

   (b) challenge it ("so what") to the point of requiring and achieving some quantification, and

   (c) as regards the concluding affirmative, do not hesitate to say "nec sutor ultra crepidat", as the consequence and decision are part of an overall Civil Engineering optimization, and should be so assessed.

   As a result a transcending design idea and action may well be the solution.

2. The problems of soil mechanics or rock mechanics and the concomitant analyses are weapons and are not the battle; they can go for or against you; the goal is a Civil Engineering decision on behalf of a Civil Engineering project, and not the subjugation of a Civil Engineering project to the imposition of soil mechanics. Many a specialization's conventional or standard practice claims on precedent have no connection with Civil Engineering reality; moreover, Civil Engineering reality is not predetermined as regards means, and ingenuity arises principally in creative revision so that a problem may be minimized or even transformed into indifference or profit. Truly, "reality" is largely what the true Engineer wishes it to be, or as he develops it. As a design Engineer, you hold the reins, and so if you fear that you may not, change the design so that you do, and let the soil scientist pursue the problem for his functional delight.

3. Soil Mechanics index parameters and much-quoted intuition are presently far from satisfactory for any design decisions. To begin with, the degree of correlation between index parameters (classification and identification data, etc.) with fundamental parameters (compressibility, resistance, permeability, erodability) is too vague. Moreover, as regards the body of the dam, the differences between different soils are attenuated and may even tend to be opposite to that which was anticipated (since compaction is a very traumatic experience to which every soil is forced to "get equivalently accustomed". A comparison may be made to the recruits trained to be marines). Guard against the bane of "consultants" who would put their "intuitions" and "experience" above, rather than at the service of, an engineering procedure of analysis and synthesis.

4. Precedents and published designs are naturally offered with the connotation that they were predominantly right, but they should be viewed and reviewed as always insufficiently right or partially wrong. From each case we try to learn something, which means that were we faced with the very same problem again, we would want to alter at least some details. Even if everything went perfectly, we should be dissatisfied that we have failed to tend to the professional call and society's needs, and we should accordingly test a little further the frontiers of impunity.

   It is philosophically unacceptable to presume to have at some time reached such a culmination of knowledge that we exclude our successors from the calling and from the need for further revision or development.

5. Non-analysed details of the overall dam structure are what really constitute the problems in the design. For obvious pedagogical reasons it is the analysable factors that are taught, published and debated with umpteen variations on a theme. Behaviour along discontinuities is our principal concern and this is where the water has a specialized capacity in the game of hide and seek, and where shearing or tensile movements find their disparaging preferences. So we must attempt to quantify the consequences of discontinuities; even though presently our technology only teaches us the behaviour of the continuum.

These thoughts have grown gradually in the process of experience suffered over 24 years, and more than twice as many dams. An attempt will be made to summarize cursorily these experiences through a critical description of the principal cases. I will discuss cross-sections, materials and specifications, foundation problems, instrumentation and observation, and finally, behaviour.

From those among the readers who may find themselves sharing my views, and therefore deriving less profit from the intended lessons, the solace I offer is that similar conclusions have been reached by way of really independent experience. Moreover, it may be of interest to note that such experience pertains to a vast sub-tropical area with which many names and adjectives of more dubious repute have been associated in Soil Mechanics (residual soils, saprolites, expansive clays, porous clays, laterites and so on). On the other hand, to those who may deem our findings different enough to require a reference to the authority they represent, it may be of interest to list some valid reasons why embankment dam engineering in Brazil has been granted special opportunities and challenging problems.

a. Firstly, there are no codes and no traditions and no precedents which condition or restrict the decision and action-making process.

b. Brazil is a young country with a very rapid growth rate. It has been forced to build a tremendous amount suddenly, starting from scratch, and to some extent forced to begin with the biggest jobs in order to make up for a time of earlier dormancy.

c. There is a very, very open attitude of international free-for-all with an eager acceptance of every school of thought at face value, at least for a first try, but with a sensitive recognition of the distinction between groups that are truly international and those that are subconsciously exporting their own nationalism.

d. In as far as Civil Engineering and especially Geotechnical Engineering are conditioned by geography, it can be said that Brazil is amongst the pioneering sub-tropical areas which face necessary adaptations.

e. Finally, there has developed a rapid recognition, proved by experience, that a high percentage of our avidly sought import of technical know-how turned out to be the unabashed import of international "don't-know-how".

## A CURSORY VIEW OF SOME CASES AND LESSONS

The Vigario Dam and Dyke (cf. Figure 1) used conservatively flat slopes. It appears evident that the vertical chimney filter was conceived principally for the purpose of the control of construction pore pressures, and for the exclusion of the flownet pore pressures from much of the downstream slope stability zone. Note the filter wells (roughly three inches every two to three metres) which extend the drainage curtain into the weathered and pervious top of sound rock.

On the other hand, we cannot fail to note, if we take into consideration several other aspects of the design, that the position adopted for the filter wells is a poor one. There is a considerable shortening of the seepage path, and the location is of little use for piping protection, expecially if we recognise that it invites localized internal erosion and possible clogging. Finally, there was no provision made for observing these filter wells and there is virtually no opportunity to maintain or reinforce the protective measures adopted.

The dominant concern in this design was stability. However, no stability computations were run or required. Note that in the dyke the downstream position of the chimney was compensated for by a higher rockfill toe.

There were no flownets prepared or required, and no seepage computations or estimates were made. The routine grouting of a curtain that had been considered in the first design was cancelled by Terzaghi in view of his evaluation of the granitic-gneiss as impervious. The dyke and dam seepages were approximately two and five litres per second. In the dam the upstream position of the chimney filter is coupled with a very short leg of an L, and water drainage is provided by pipes (8"and 12" cast iron). In the dyke the sand drainage blanket extends over the entire base, and cast iron pipes within the blanket provide additional drainage.

Finally, there were no compressibility estimates made nor was there any concern about differential settlements of the abutments, despite the upper horizon of porous clay (cf. Santa Branca, Figure 2, and Tres Marias, Figure 12). The chimney filters penetrated 5m normally into the abutments and intercepted the preferential contact flow.

In the Santa Branca Dam it was recognized that the principal seepage comes from the foundations.

Careful water pressure tests according to the best techniques of the time indicated the need for a shallow grout curtain. This was meticulously installed and yielded some early information (cf. V.F.B. de Mello et al, "Some quantitative investigations of curtain grouting of rock foundations of early dams", First Panam. Conf. SMFE, Mexico 1959, vol. II p.699) on the distinction between groutability and the need for grouting. Note that such earlier thinking has

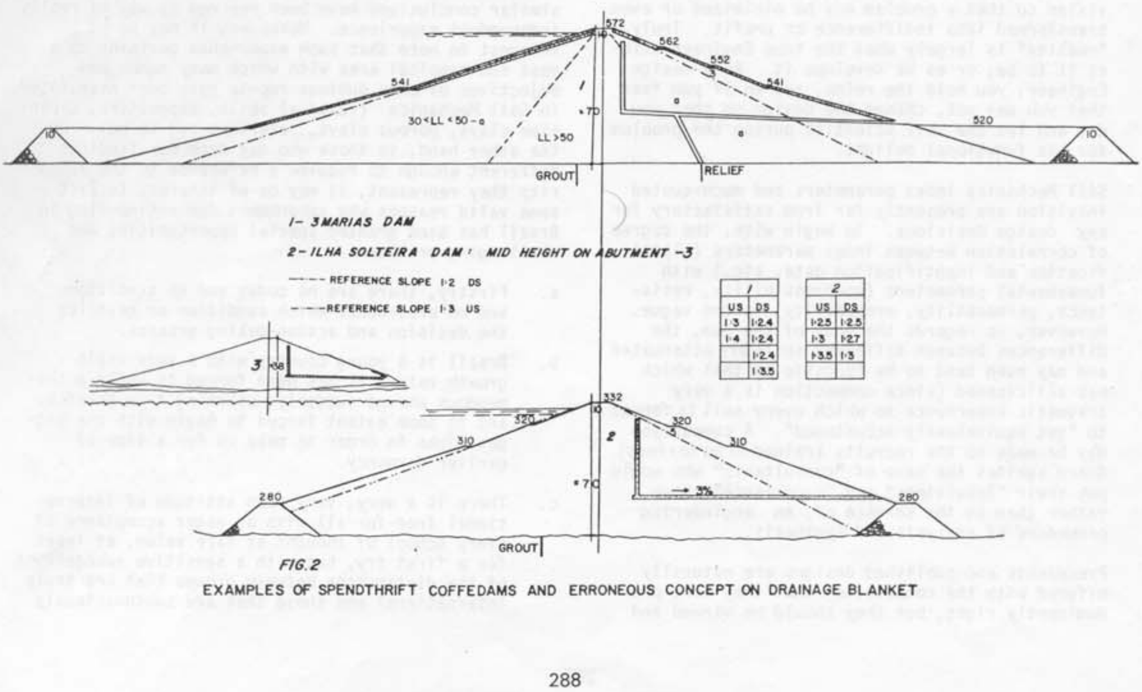


FIG.2

EXAMPLES OF SPENDTHRIFT COFFEDAMS AND ERRONEOUS CONCEPT ON DRAINAGE BLANKET been superceded (cf. V.F.B de Mello, "Discussion", Montreal 1965, 6th ICOSOMEF, vol. III p.577).

Stability analyses led to the use of steeper slopes, and the use of a gradually steepening slope towards the upper reaches. The height downstream is 20m and the slope is about 1:1,5. This arrangement behaved perfectly satisfactorily.

The grassed slopes were interrupted by berms at about 10m elevations. This gave an apparently optimum bahaviour which minimized infiltration and interrupted the kinetic energy of the runoff before the erosion started. Flatter slopes increased the infiltration and the wetting-drying damage (in our climate there isn't the additional problem of freeze-thaw). The chimney filter should, in principle, not have stopped so far below the normal high water level, although in this instance there was no undesirable seepage above the chimney as would possibly occur with predominantly horizontal flow.

The earth dam design was an entirely independent task. The rockfill cofferdams were later fitted in by the hydraulic engineer, with no attempt to revise slope angles or to optimize the use of rock. The base of the dam could have been shortened if the task had been viewed as a joint problem.

Stability analyses were dominated by the expected high construction pore pressures ($B \approx 30\%$), and the imposed factor of safety of 1,5 which was in accordance with the accepted practice of the time. Moreover, for the upstream slope the factor of safety was dominated also by the flownet for the total drawdown case and by the lowest strength which had been calculated from an isotropic consolidated quick "saturated" triaxial test at the one extreme and by a slow test at the other. The two factors of safety gave an average of at least 1,5. The chimney filter introduced a remarkable improvement in the drawdown flownet. Moreover, it is important to ponder that academic developments now permit better or even absolute saturation of test specimens by backpressure, with a consequential reduction of the consolidated quick strength (is it realistic or is this merely more stringent?). The gist of all these changes, together with developments in the computation hypotheses and procedures, have very greatly affected the would-be precedents for upstream slopes.

The support of abutments by porous clay horizons was studied in particular with respect to differential settlements in 1957, but no measures were required because of the reasoning that a one in two hundred differential settlement, which just cracks brick-wall panels (Skempton and MacDonald 1956), should not begin to crack a compacted clay fill.

With regard to the construction pore pressures observed, bona fide results indicate $B = u/\gamma Z$ of less than 20 per cent. Surface indicators revealed negligible movement after each elevation had been reached. This dam involved the first use of micaceous residual soils in the compaction process, principally downstream of the chimney. The borrow rejection criterion used involved the following observations: if the material in the Proctor compaction mould was found to expand, the material was rejected. The field compaction rejection criterion employed was the usual one, that is, that the compaction should be equal to or greater than 95% Proctor.

Figure 2 exemplifies two engineering mistakes that have frequently occurred. At the Tres Marias Dam (1958 - 1960) the uneconomical position of the cofferdams and the subsequent filling in of a "random zone" reduced the effective height of the dam. At Ilha Soltiera (1969 - 1973) this situation is attenuated.

In both cases, however, the significant misconception concerned the horizontal drainage blanket. When once it is recognized that practically all the seepage and the piping is concerned with the upper horizon of the foundation, the obvious action is to place the filter drainage blanket directly on the foundation. The hydraulics of the minimal flow across the compacted embankment does not have to rely on the elevation head, neither should it be conditioned by an exit elevation with respect to the very rare highest tailwater elevation. Three or four major dams oriented and designed in this erroneous manner, developed at the abutments, on first filling, a considerable downstream uplift, flow and incipient piping which had to be corrected rapidly by a blanket of rock spoil, two to three metres thick.

The Tres Marias Dam presented some interesting novelties, more because of the different specialists involved than because of differences in materials, construction specifications, equipment, strength envelopes and end-of-construction stability computations. These were issues of controversy and involved important decisions. Side issues were the use of vertical as opposed to inclined filters, the degree of filter compaction and so on, and these will not be mentioned here. A very important problem was the foundation behaviour on compressible porous clay.

As this dam tended to embody a second milestone in our earthdam engineering, we shall return to it later and pay more attention to specific details. However, let us first proceed to discuss some cross-sections which exemplify the overall design details. Figure 3 shows three cases of dams fifty to sixty metres high which were designed as a sequel to Terzaghi's contribution.

Saramenha (1956 - 1958) was considered a difficult case because of a deep V-shaped valley and the lack of locally available materials except a deeply weathered metamorphic body which yielded a slightly clay silt having a small micaceous content. The chimney filter was inclined as this increased the seepage distance without aggravating the downstream pore pressures when there was a full reservoir. The inclined filter was really built in steps by trenching. Weathered rock from the spillway and diversion conduit excavations permitted a downstream rockfill toe which improved the stability. The overall layout design by an hydraulic engineer did not permit an optimized upstream cofferdam and diversion section. The double V topography should be considered a most unhealthy situation for a silty material in view of our present-day prognostications as to cracking. However, the performance of this dam has been excellent and one cannot fail to point out that the thickness of the overall impervious section has a considerable bearing on the problem, and so also the occasions and rates at which the settlements take place for these could lead to cracking.

The Nhangapi dyke (7 million cubic metres) shows the purely soil mechanics cross-section which was developed for a case where absolutely no rock or granular materials were available.

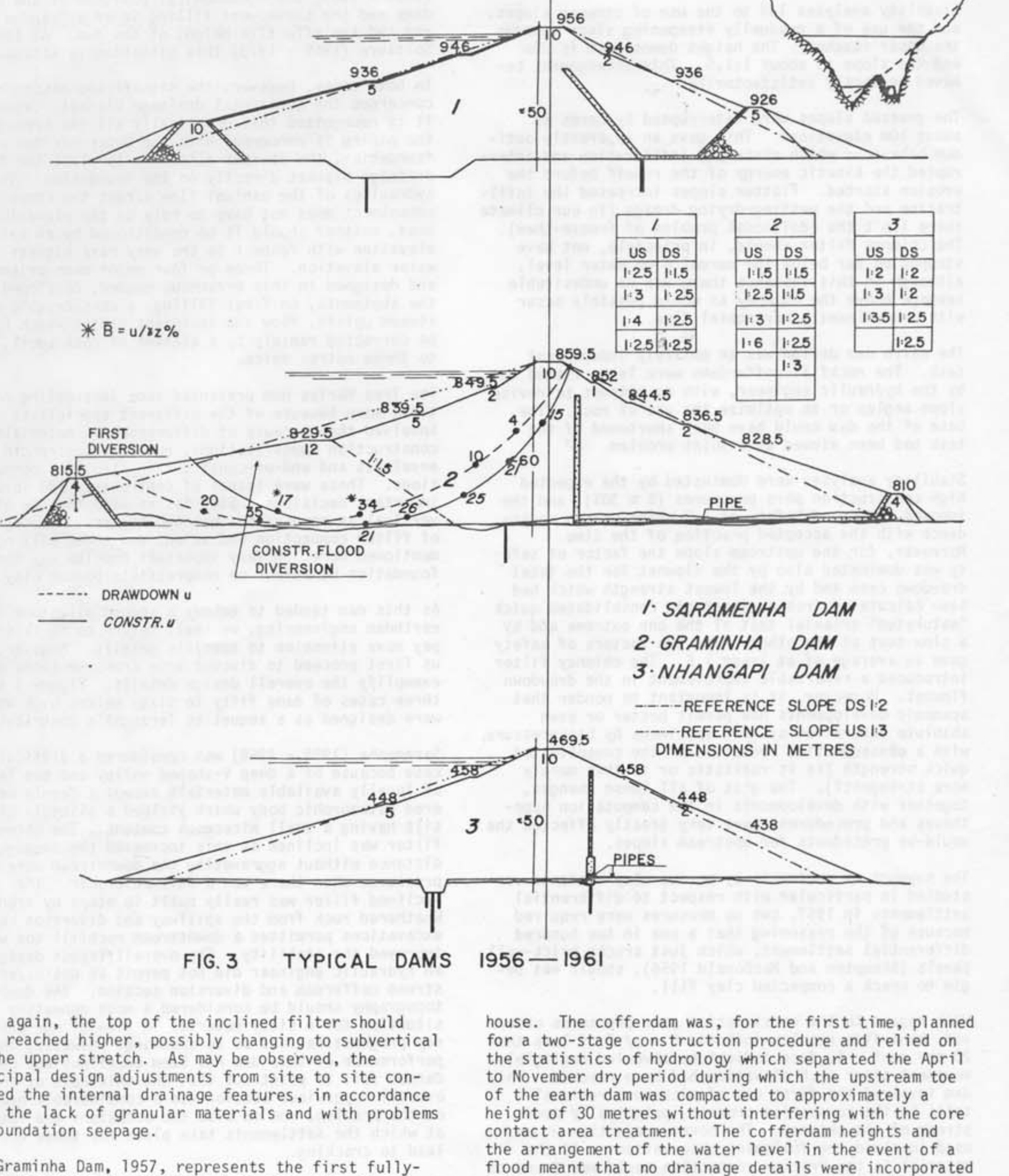


FIG. 3 TYPICAL DAMS 1956—1961

Once again, the top of the inclined filter should have reached higher, possibly changing to subvertical in the upper stretch. As may be observed, the principal design adjustments from site to site concerned the internal drainage features, in accordance with the lack of granular materials and with problems of foundation seepage.

The Graminha Dam, 1957, represents the first fully-studied attempt at optimized Soil Mechanics in conjunction with an overall hydraulic layout. The hydraulic structures were fitted into very compact arrangement which involved a single tunnel and shafts, a morning glory spillway and an underground powerhouse. The cofferdam was, for the first time, planned for a two-stage construction procedure and relied on the statistics of hydrology which separated the April to November dry period during which the upstream toe of the earth dam was compacted to approximately a height of 30 metres without interfering with the core contact area treatment. The cofferdam heights and the arrangement of the water level in the event of a flood meant that no drainage details were incorporated into the compacted clay dam section which worked temporarily as a cofferdam.

The clay material used in the dam had a dry density of 1,35 tonnes per cubic metre and an optimum water

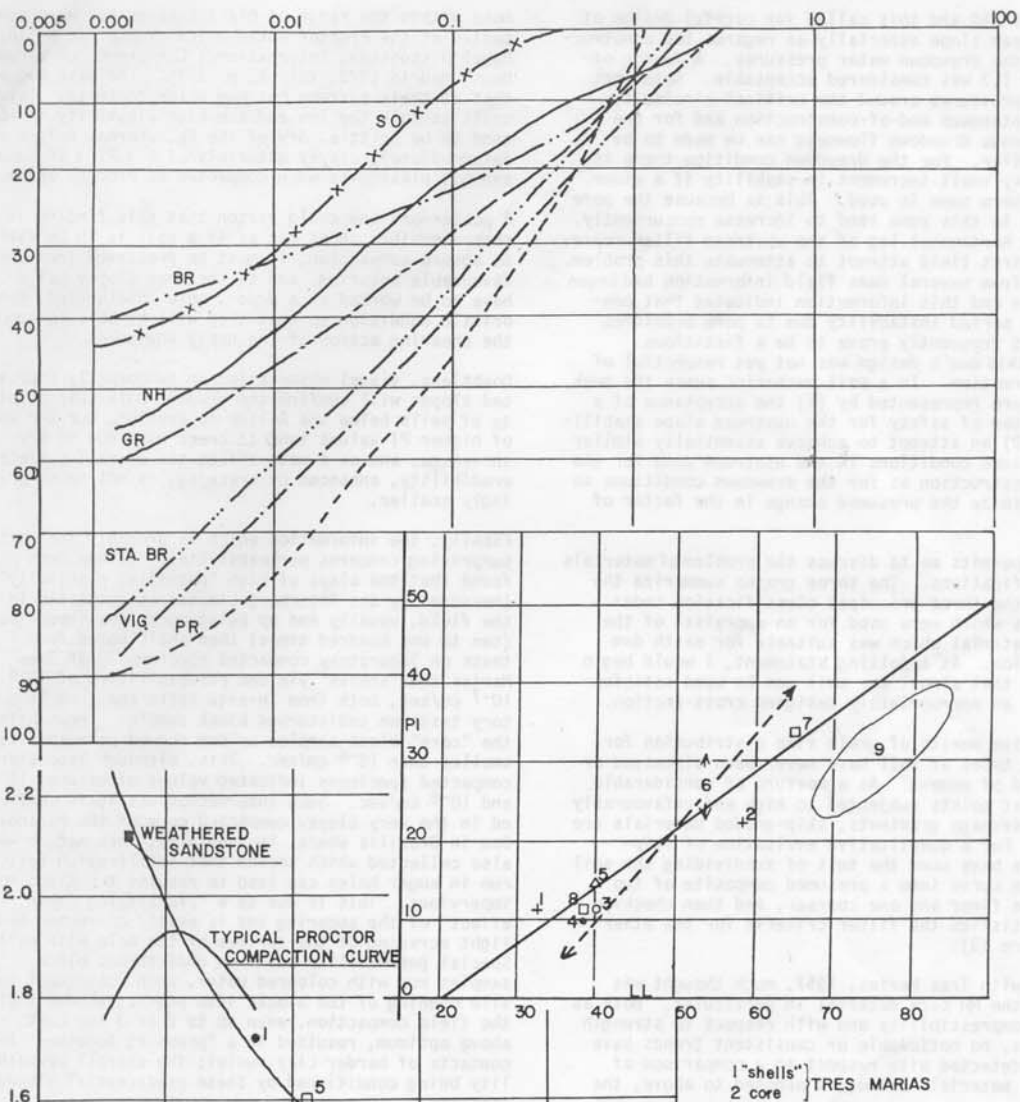


FIG 4 IDENTIFICATION DATA ON TYPICAL COMPACTED SOILS

content of 35% and this called for careful design of the upstream slope especially as regards the construction and the drawdown water pressures. A factor of safety of 1,3 was considered acceptable. Note that the pore pressures around the critical circles for the instantaneous end-of-construction and for the instantaneous drawdown flownets can be made to be quite similar. For the drawdown condition there is only a very small increment in stability if a wider upstream berm zone is used. This is because the pore pressures in this zone tend to increase concurrently. The short horizontal leg of the upstream filter represents a first timid attempt to attenuate this problem. Although from several dams field information had begun to come in and this information indicated that construction period instability due to pore pressures would most frequently prove to be a fictitious problem, this dam's design was not yet respectful of such information. In a soil mechanics sense the meek advances are represented by (1) the acceptance of a lower factor of safety for the upstream slope stability, and (2) an attempt to achieve essentially similar pore pressure conditions in the upstream zone for the end-of-construction as for the drawdown conditions so as to minimize the presumed change in the factor of safety.

Figure 4 permits me to discuss the problem of materials and specifications. The three graphs summarize the data for the three principal classification index parameters which were used for an appraisal of the type of material which was suitable for earth dam construction. As a jolting statement, I would begin by saying that almost any soil can be used satisfactorily in an appropriately designed cross-section.

The relative merits of grain size distribution for different types of soil have never been discussed or considered of moment. As a measure of considerable prudence at points subjected to high and unfavourably directed seepage gradients, skip-graded materials are avoided. For a quantitative evaluation of skip-grading we have used the test of subdividing the soil grain size curve into a presumed composite of two soils, one finer and one coarser, and then checking if one satisfies the filter criteria for the other (see Figure 13).

Starting with Tres Marias, 1957, much thought was given to the MH core material in particular. Both as regards compressibility and with respect to strength parameters, no noticeable or consistent trends have yet been detected with respect to a comparison of compacted materials below, as opposed to above, the A-line.

Soils above the A-line were claimed to be more capable of "plastic adjustment" to differential deformations, and therefore less susceptible to cracking. If we judge that the plastic limit, PL, is a rough measure of the water content for an equal susceptibility to cracking, (at zero confining stress), we have schematically compared the Proctor compaction optimum water content with the plastic limit of the soil. Borrow area investigations have been routinely conducted by the Hilf-Proctor technique and two important plots have been made of the natural versus the optimum water content and the plastic limit versus the optimum water content.

The available data have been put together in the graph of PI versus the optimum water content with the intention of checking if soils of higher plasticity do fall more within the range of plastic behaviour when compacted at the Proctor optimum (cf. V.F.B. de Mello, Panel Discussion, International Conference on Large Dams, Madrid 1973, Vol. V, p. 394). The data suggest that at their extreme optimum water contents, clayey soils at both the low and the high plasticity indices tend to be brittle, dry of the PL, whereas only the intermediately clayey materials of $6 \gtrsim PI \gtrsim 25$, would exhibit plasticity when compacted at Proctor optimum.

A posteriori one could reason that this finding is comprehensible in as far as if a soil is to be able to absorb compaction, it must be preserved from unfavourable shearing, and so the more clayey soils have to be worked at a dryer, more unsaturated, more brittle condition so that they will be able to support the shearing action of the heavy equipment.

Doubtless, visual observation on temporarily unprotected slopes will confirm the greater bulk susceptibility of soils below the A-line to erosion, but the soils of higher PI values tend to crack more due to dry shrinkage, and as a nett effect the overall surface erodibility, enhanced by cracking, is not so surprisingly smaller.

Finally, the information which is probably the most surprising concerns permeabilities. It has been found that the clays of high "potential plasticity" (measured by the Atterberg Limits) as compacted in the field, usually end up being much less impervious (ten to one hundred times) than anticipated from tests on laboratory compacted specimens. At Tres Marias the "shells" yielded permeabilities of $10^{-6}$ to $10^{-7}$ cm/sec, both from in-situ tests and from laboratory tests on undisturbed block samples. Meanwhile the "core" block samples seldom showed permeabilities smaller than $10^{-6}$ cm/sec. This, although laboratory compacted specimens indicated values of between $10^{-7}$ and $10^{-8}$ cm/sec. Such information was again confirmed in the very clayey compacted core of the Paranoa Dam in Brasilia where, incidentally, information was also collected which showed that infiltration tests run in auger holes can lead to results ten times more impervious. This is due to a "plastifying smear effect" of the augering and is easily corrected by a light scraping of the surface of the hole with nails. Special permeability tests on undisturbed block samples run with coloured water, upon subsequent tensile opening of the blocks into pieces, showed that the field compaction, even up to 2 or 3 per cent above optimum, resulted in a "pressing together" into contacts of harder clay nuclei; the overall permeability being conditioned by these preferential sinuous contacts.

Although the permeabilities in question are themselves beyond discussion, it is the relative permeabilities that determine flownets and therefore the problem becomes important. It appears from block samples that in the sandier soils it takes a change of about two hundred per cent in the value of the void ratio, e, to cause a tenfold decrease of permeability. This is one cycle in a semilog straight line plot: for instance, a $\Delta e$ of about 0,07 in a clay compared to about 0,20 in a slightly clayey sand leads to a one cycle change in the value of the permeability coefficient.

Such changes of permeability due to compression have not been considered in textbooks or publications, nor has the effect of such changes in altering the flownets across the embankment been considered. However,

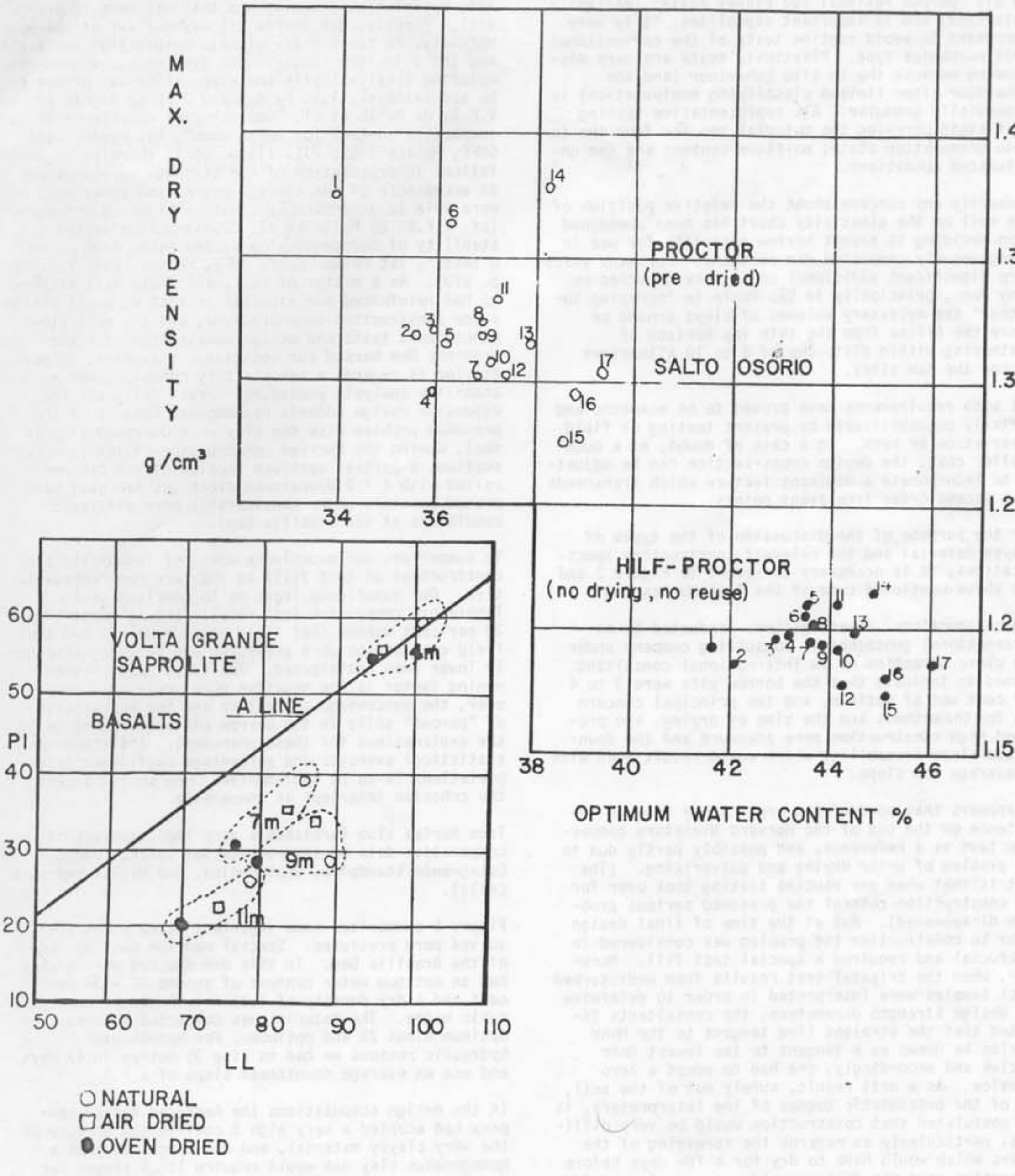


FIG 5 ROUTINE TEST PECULIARITIES (compaction and plasticity)

these do have a significant influence in pushing the phreatic line upwards. Figure 5 shows, however, that in all "porous residual red clayey soils" (partially laterized) and in important saprolites, it is very important to avoid routine tests of the conventional soil mechanics type. Plasticity tests are very misleading because the in-situ behaviour (and the behaviour after limited plastifying manipulation) is essentially granular. All representative testing must avoid carrying the material too far from the in-situ granulation state, moisture content and the unsaturated conditions.

Presently any concern about the relative position of the soil on the plasticity chart has been abandoned when deciding to accept borrow materials for use in homogeneously compacted dam sections. For many years very significant additional costs were incurred on many dams, principally in São Paulo in "scraping together" the necessary volumes of clays around or above the A-line from the thin top horizon of weathering within distances of 8 to 10 kilometres around the dam sites.

All such requirements have proved to be academic and strictly unquantifiable by present testing or field observation or both. In a case of doubt, at a much smaller cost, the design cross-section can be adjusted to incorporate a dominant feature which transcends such second order irrelevant points.

For the purpose of the discussion of the types of clayey material and the relevant construction specifications, it is necessary to revert to Figure 2 and the above-mentioned case of the Tres Marias Dam.

Preconstructional investigations conducted by an international geotechnical consulting company under the close direction of an international consultant seemed to indicate that the borrow pits were 3 to 4 per cent wet of optimum, and the principal concern was for the methods and the time of drying, the presumed high construction pore pressure and the downstream slope instability which would result even with an average 1:3 slope.

It appears that part of the error arose from an insistence on the use of the Harvard Miniature compaction test as a reference, and possibly partly due to the problem of prior drying and pulverising. (The fact is that when our routine testing took over for the construction control the presumed serious problems disappeared). But at the time of final design prior to construction the problem was considered to be crucial and required a special test fill. Moreover, when the triaxial test results from undisturbed block samples were interpreted in order to determine the design strength parameters, the consultants insisted that the straight line tangent to the Mohr circles be drawn as a tangent to the lowest Mohr circles and accordingly, one had to adopt a zero cohesion. As a nett result, surely not of the soil, but of the pessimistic dogmas of the interpreters, it was postulated that construction would be very difficult, particularly as regards the spreading of the layers which would have to dry for a few days before compaction, and the design would even require sandwich horizontal drainage blankets at every 10m of elevation (cf. Selset Dam, Geotechnique). The problem was of great importance because the 600 000$m^3$ of sand required would be more than doubled and the sand had to be trucked from a minimum distance of about 90 kms.

To cut short a long story, it may be emphasized that our data and impressions supported a totally different, optimistic situation and this was made to prevail. Firstly, the borrow pit was not wet of optimum; secondly, in the hot dry climate evaporation was rapid and the principal construction problem was a need for watering (statistically the evaporation was proved to be approximately 1,5% by day and 0,7% by night: cf. V.F.B. de Mello et al, "Geotechncial esperience in compaction control for earth dams", 1st Panam. Conf. SMFE, Mexico 1959, Vol. II, p. 637); thirdly, by statistical interpretation of the strength envelopes and an acceptance of the average regression equations, we were able to demonstrably count on higher Q strengths (cf. V.F.B. de Mello et al, "Construction period stability of homogeneous compacted earth dams using Q tests", 1st Panam. Conf. SMFE, Mexico 1959, Vol. II, p. 673). As a matter of fact, all field data hitherto had reinforced our expectation that we would obtain a low construction pore pressure, and the meticulous concomitant tests and design computations for the Graminha Dam backed our optimistic viewpoint, in particular as regards a satisfactory cohesion, and a stability analysis procedure. Practically all the expensive design addenda recommended because of the presumed problem with the clay were disposed of. In fact, during the hurried construction of the closure section, a partial upstream section of the dam was raised with a 1:2 downstream slope (as had just been proved possible under considerably more difficult conditions at the Brasilia Dam).

To summarize, our experience does not favour the preconstruction of test fills as they are non-representative. Our experience leads us to conclude that laboratory compaction test results are at least 10 - 20 per cent weaker that field block samples, and that field constuction pore pressures are very significantly lower than anticipated. The first major intervening factor is the negative pore pressure; moreover, the percentage saturation and the macro-pores of "porous" soils in the borrow pit are assumed to be the explanations for these phenomena. Insistance on statistical averages and percentage confidence interpretations is to be implemented. One should accept the cohesion intercept as inexorable.

Tres Marias also furnished a very important set of comparative data on instrumentation (USBR, London, Casagrande standpipe, Geotechnica, and Maihak pressure cells).

Figure 6 summarizes some important cases which involved pore pressures. Special mention must be made of the Brasilia Dam. In this dam the red porous clay had an optimum water content of around 30 - 34 per cent and a dry density of 1,35 - 1,40 tonnes per cubic metre. The material was compacted between optimum minus 2% and optimum. For hydrologic-hydraulic reasons we had to rise 35 metres in 42 days and use an average downstream slope of 1:1,4.

In the design computations the American design company had adopted a very high B coefficient because of the very clayey material, and had concluded that a homogeneous clay dam would require 1:3,5 slopes for stability. Accordingly, the earth-rock section had been adopted as imperative, and this despite extreme difficulties and the cost of quarrying in the local geology. When construction had been in progress for a few months, a casual check on the capacity of the diversion flume led us to conclude that it would be impossible to avoid overtopping of the dam during the

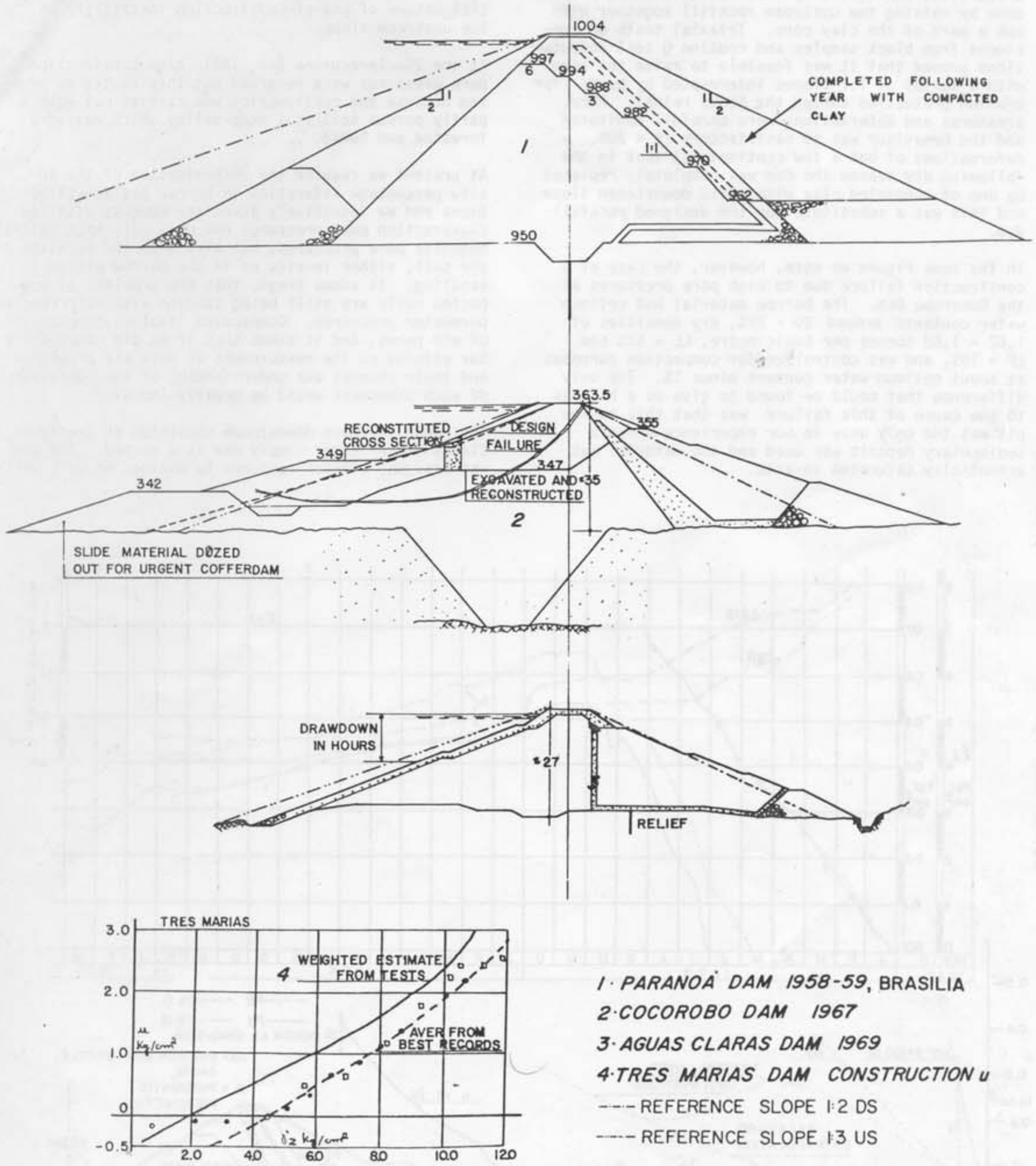


FIG 6 CRITICAL CONDITIONS ON PORE PRESSURES

season of heavy rains and flood unless the spillway crest level was hurriedly reached; this could only be done by raising the upstream rockfill together with but a part of the clay core. Triaxial tests on specimens from block samples and routine Q test computations proved that it was feasible to raise the core with a series of 1:1 slopes interrupted by berms (for erosion protection during the heavy rains). Pore pressures and deformations were carefull monitored and the behaviour was so satisfactory (B < 20%, deformations of but a few centimetres) that in the following dry season the dam was completely replaced by one of compacted clay with a 1:2 downstream slope and this was a substitute for the designed rockfill dam.

In the same Figure we note, however, the case of a construction failure due to high pore pressures at the Cocorobo Dam. The borrow material had optimum water contents around 20 - 22%, dry densities of 1,62 - 1,68 tonnes per cubic metre, LL = 57% and LP = 19%, and was controlled for compaction purposes at about optimum water content minus 1%. The only difference that could be found to give us a hint as to the cause of this failure was that this borrow pit was the only case in our experience where a sedimentary deposit was used and the material was essentially saturated in-situ.

The failure zone was easily reconstructed in about 15 days. This confirms the relatively inconsequential nature of end-of-construction instability in the upstream slope.

At the 25m Saracuruna Dam, 1961, high construction pore pressures were recorded but they caused no problem because the construction was carried out with a partly porous soil in a deep valley which was very forested and humid.

At present we require the determination of the in-situ percentage saturation in borrow pit investigations and we intuitively associate success with low construction pore pressures not only with high initial negative pore pressures, but also with the aeration of the soil, either in-situ or in the borrow pit or in handling. It seems tragic that the problems of compacted soils are still being studied with reference to porewater pressures. Compaction involves compression of air pores, and it seems that if we did concentrate our efforts on the measurement of pore air pressures and their changes our understanding of the behaviour of such phenomena would be greatly improved.

The rapid drawdown downstream condition at the Aguas Claras (1972) water supply dam is a record. The dam was designed without recourse to borings or soil tests.

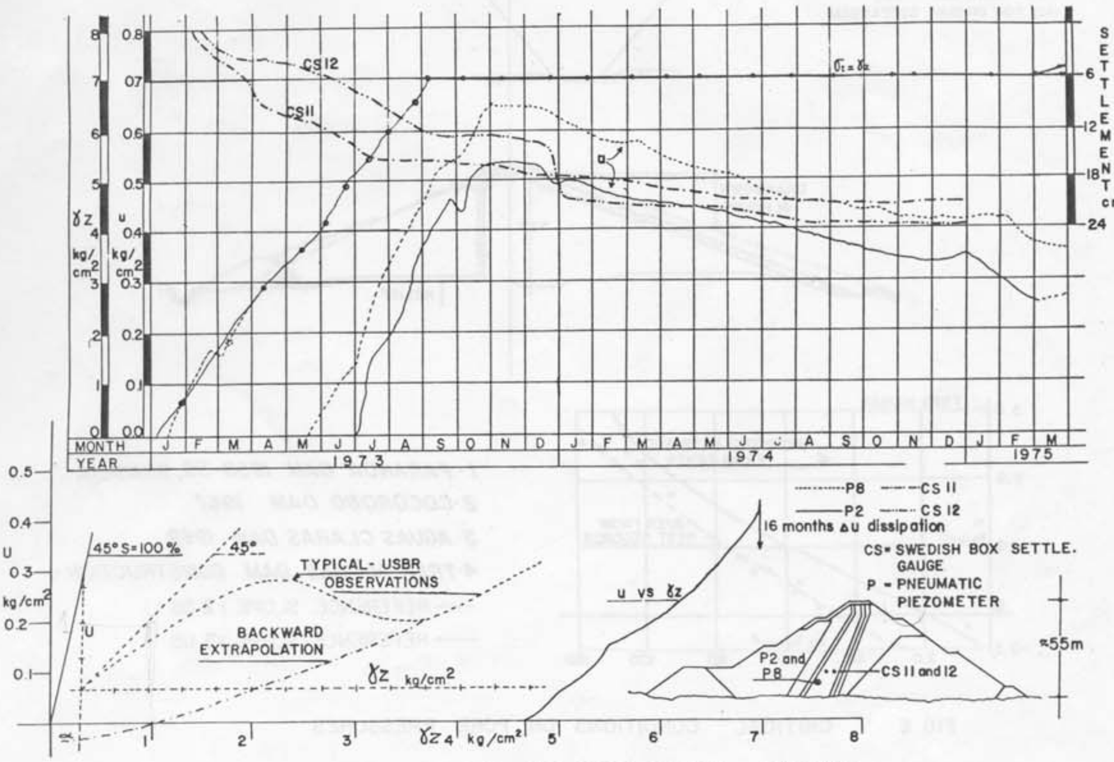


FIG 7 MOST CLAYEY MATERIAL AND BEST CONSTR.u RECORDS

It is felt that the drawdown condition conventionally assumed in Soil Mechanics practice is very pessimistic since in most dams during the first few years of operation the water level goes up and down in cycles, and only gradually goes lower and lower in a succession of dry years with the full operational load.

Meanwhile, when drawdown first occurs, the percentage of air in the compacted and as yet unsaturated soil is sufficient to minimize the problem due to the compressibility of the pores since the material is really in a recompression condition. Figure 7 records the surprisingly favourable results in the Salto Osorio Dam (1971 - 1974). The material of which this dam is constructed is certainly the most clayey we have used to date. It can be described as a red porous soil of optimum water content of approximately 40%. Much of it was compacted between optimum and optimum plus 3%; the borrow pit was at about 10% above optimum. The settlement records tally with the pore pressure data which show a very much smaller field compressibility than was determined from laboratory compacted specimens. A very high percentage of the settlement is instantaneous. It may be noted that the soil is not a swelling material since the oedometer swelling index is only of the order of 10 - 15% of the compression index. The settlement appears to be of the order of one third or less of that computed from oedometer tests on block samples, and even less when compared with laboratory compacted specimens.

In these soils it has been found necessary to restrict the loads and the tyre pressures of all the standard earth-moving equipment in order to avoid excessive lamination. Figure 8 shows a recent attempt at improving the statistical presentation of compaction data from the Hilf inspection tests. Earlier presentations of separated frequency curves on percentage compaction and on water contents did not permit a visualization of the influence of the latter on the former.

It must be recalled that it was shown statistically (cf. V.F.B. de Mello et al, "True representation of the quality of a compacted embankment", 1st Panam. Conf. SMFE, Mexico 1959, Vol. II, p. 657) that despite careful quality control and inspection testing it is impossible to exclude from the fill the occurrence of areas compacted below the specified densities. Therefore the consequences of such occurrences have to be investigated. On the one hand it is a problem of statistics, on the other hand, there is the extreme hypothesis of a systematic concentration of such occurrences which may well constitute a layer. Thus, one of the very important problems faced in every single project has been that of the consequences of layers or sub-layers that may exist within the dam. Publications are lacking on the properties of fills compacted somewhat below the conventionally cited criteria.

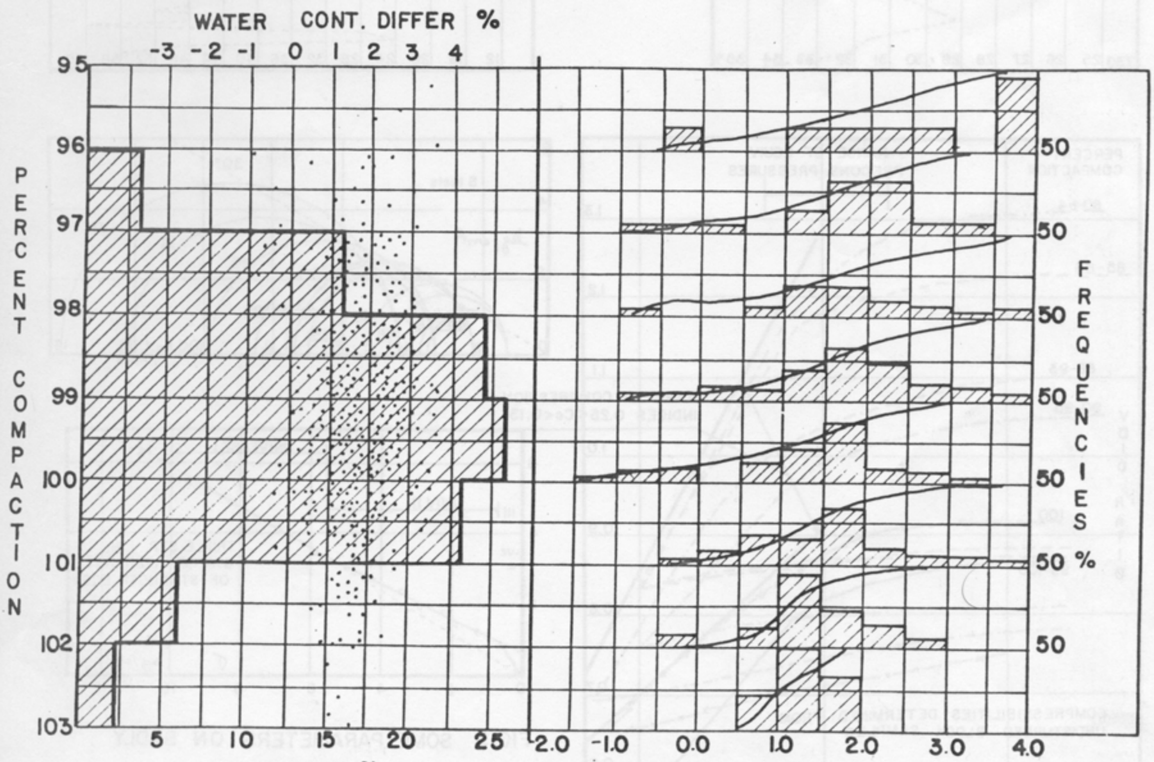


FIG. 8 SALTO OSÓRIO COMPACTION STATISTICS (467 tests)

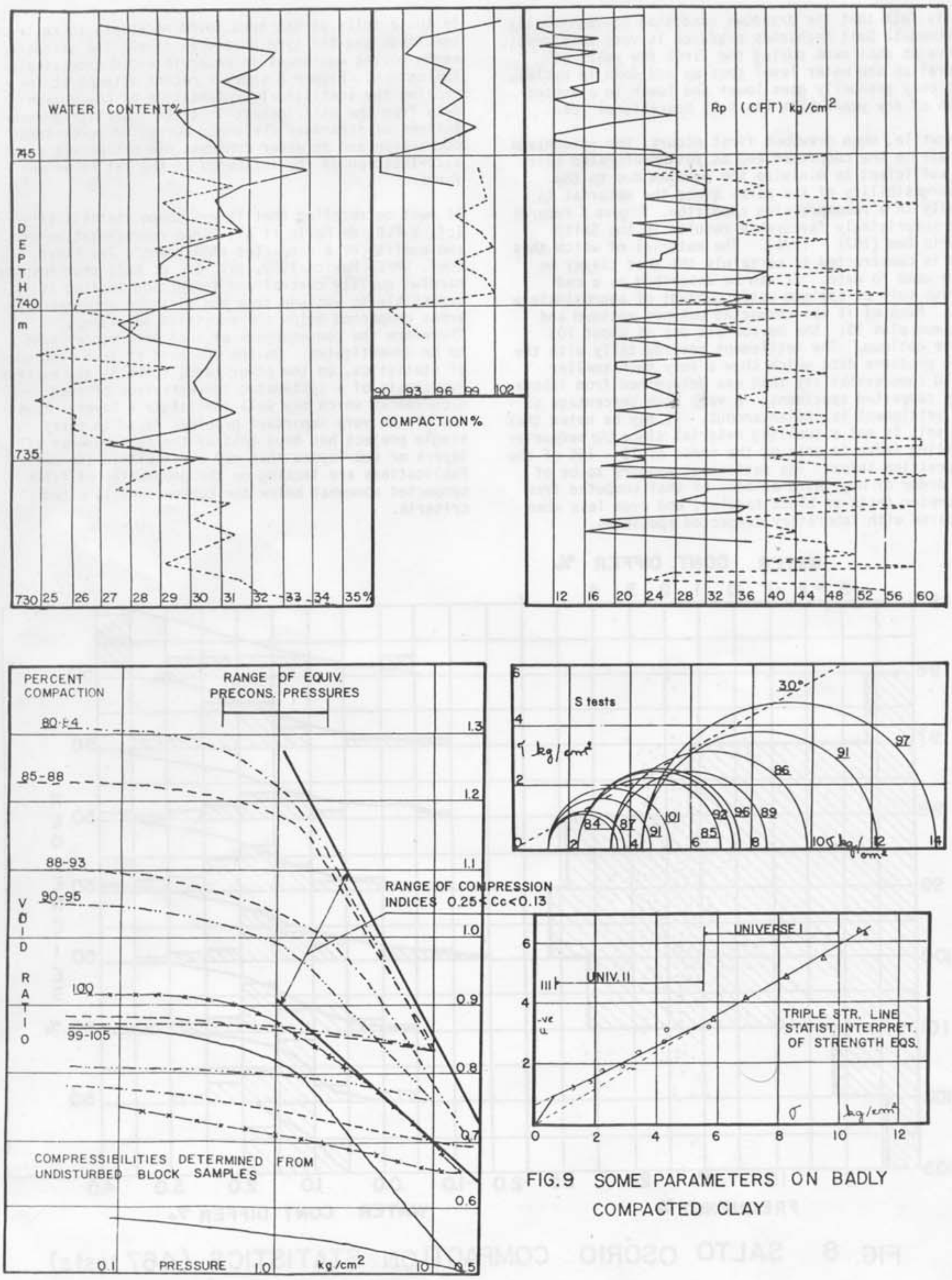


FIG. 9 SOME PARAMETERS ON BADLY COMPACTED CLAY

Figure 9 summarizes the results from Juqueri Dam, 1972. This is a 30m dam which uses a very clayey material. The compaction was presumably controlled most meticulously. As usual, the first hint of a problem arose due to a "total water loss" in drilling for standpipe piezometers; and, as usual, the immediate hint was of "cracks".

A meticulous investigation included 25 Dutch cone penetration tests (Point Resistance $R_p$) and 10 adjacent test pits and these proved that there were sub-layers which were essentially uncompacted.

It can be seen that both as regards the strength parameters and as regards the compressibility, the change from 98% compaction to 90% compaction of a material is a question of degree, and not a dramatic distinction of black and white.

The principal points to make with regard to Figure 9 are :

(a) Firstly, the static cone penetrometer point resistance is a sensitive means of detecting looser sub-layers, somewhat in the fashion of the old Proctor Needle, although water content deviations from the optimum interfere significantly with the results. In cases of doubt, I suggest the use of the Hilf technique which is associated with a miniature compaction test, in particular because of the specimen size which is a collateral of the investigation of the moisture deviation.

(b) Secondly, in both the oedometer and the triaxial results the percent compaction reflects predominantly a preconsolidation effect, which incidentally, checks very closely with tyre pressures and pressure bulbs. A 96% compaction corresponds to about $4kg/cm^2$ preconsolidation, the 90% corresponds to about $1,5kg/cm^2$ and 102% compaction to about $5,5kg/cm^2$.

(c) Thirdly, because of the preconsolidation effects it is very important to subdivide the Mohr-Coulomb strength envelope into three straight lines according to statistical universes; (i) for the virgin compression condition, (ii) for the preconsolidation condition with low overconsolidation ratios and a cohesion intercept, and finally, (iii) for the high overconsolidation ratios which tend to develop negative pore pressures when sheared.

Mention must be made of a 30m high fill of gneiss saprolites (similar to Vigario) which has been built by end-dumping into 25m of water at the Billings Reservoir. Here special precautions against aeration were taken. The tipping resulted in an essentially instantaneous compression and about 80% compaction.

As regards permeability, the variation can be ten to one hundred fold. As a result, the effect on the flownet must be assessed. Figure 10 was drawn up to investigate the influence of a thin loose layer on the important flownet which affects the downstream stability and the drainage features. The overall influence can be seen to be small, even if the full upstream-downstream path were to be crossed by the suspected layer. Generally, one tries to render such an occurrence improbable by means of requiring the construction to be carried out by means of placing the fill in patches which stretch across part of the width of the dam at its lower elevations.


FIG. 10 EFFECT OF MORE PERVIOUS COMPLETE "UNCOMPACTED" LAYER ON FLOWNET

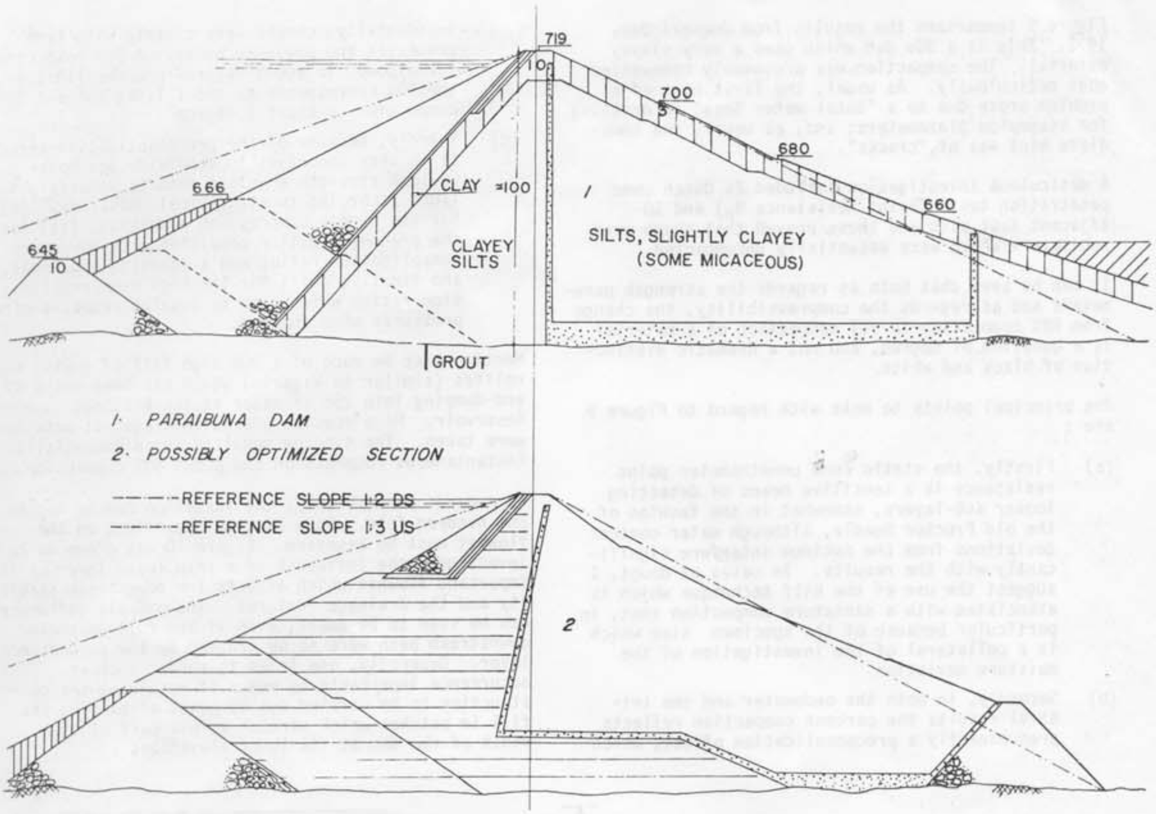


FIG 11 RECENT DAMS AND SUGGESTION, GRANITO-GNEISS

Figure 11 concerns two recent dams two and a half times the height of Vigario, and both of these employed not only the same soils but also a definitely micaceous underlying saprolite horizon.

The upper cross-section represents a recent dam which employs the same typical horizons of the advanced weathering of granite-gneiss as first employed by Terzaghi. Without implying any judgement in favour or not of the design, the fact is that it is said to have been conditioned by a topographic difficulty which required a shortening of the overall base and by significant shortages of rock and of the upper totally weathered horizon of red silty clay. The main volume of borrow was of saprolites, predominantly silts somewhat sandy and slightly clayey, some micaceous. The design decision to employ these materials was supported by voluminous test data, including careful conventional triaxial tests on block samples from the compacted fill. Field inspection was extensive and intensive, and the dam was much instrumented. The decision to employ in the external boundary of the compacted earth section the available, but limited, more clayey material, was aimed at avoiding the rain erosion before the rockfill advanced onto the upstream face or before the protection by grass and berms with drainage ditches was ready on the downstream face.

Immediately prior to the start of filling, as a routine measure of prudence, a special consultant was asked to review the case. As reported, the consultant began by stating that to his knowledge such micaceous soils had never before been used in a major dam. Moreover, despite such lack of personal knowledge on such soils as compacted (and setting aside the triaxial test data) he established on the basis of plasticity data that the parameters showed a zero cohesion, that is the cohesion intercept of the linearized equation was zero. A low residual $\phi$ value was also indicated. This low value was close to mandatory because the laminar particles would determine the preferential horizontal shear planes. Moreover, the upstream clay surfacing and the clayey silt core could crack and the chimney filter would clog, so that the downstream stability should be computed for full hydrostatic pressure which should be applied at the chimney filter. It is considered to be an impervious curtain. Finally, the downstream clay surfacing would be impervious and this would permit the build-up of considerable reservoir pore pressures in the downstream zone, since some seepage would get by the filters whether they were considered to be drainage features or they were rendered fully impervious, and this seepage would emerge from the more pervious saprolites of the abutments into the much less pervious compacted saprolites. Thereupon, as a remedy, the downstream clay surfacing should be torn up by drainage trenches. As an overall conclusion, for a computational hypothesis the designers were told to consider possible wedge failures along horizontal planes of weakness which would be subjected to uplift varying linearly from 50 per cent of full hydrostatic head at the chimney filter to zero at the downstream face. Note that neither at the upstream end of the horizontal plane was there compatibility with the stated pore pressure on the vertical surface, nor along the plane was there any compatibility with the really critical possibility that if any of the presumed emergence of seepage into the downstream zone occurred, it would be held by the downstream impervious facing.

As a nett conclusion, the downstream slope of the dam was proved to be unstable, and it was necessary to reinforce it by applying a compacted rockfill to the downstream slope of 1:2,5.

Two hydrologic factors of great relevance that would be considered in favour of postponing any premature decision of great economic impact were summarily discarded as unimportant: one was the fact that the reservoir would fill at a rate of less than about 2,5 - 5 cms/day over the upper 20m and 50m respectively; the second was that the hydraulic features are such as to permit full stoppage of filling at about 20m below crest.

Urged to intervene in this case, I accepted responsibility for dispensing with any "stabilizing" work unless performance during filling truly called for it. The micaceous appearance of the soil, which was judged by particles that adhere to the palm of one's hand, tends to give a pessimistic view. The quantitative mica count had been determined to be more than 8 000 grains in one sample, and to be about 47 per cent of the material. Unfortunately, neither were there more of such tests, nor are there sufficient studies to permit an assessment of the quantitative effects of different percentages of mica on the fundamental soil parameters.

Moreover, a gneiss - granite saprolite having micaceous particles alongside a considerable portion of much harder angular grains such as quartz would not tend to give low residual $\phi$ values, even if it were not absurd to consider such a parameter for the initial stability of a compacted fill. As a matter of fact, careful tests in special annular shear equipment revealed a neligible drop off from peak to residual, the residual value of $\phi$ being of the order of $28^{\circ}$.

The pore pressure assumptions were preposterous: to claim that the clay and silty clay compressed by the rockfill and wetted by the very slow rise of water would crack, while the downstream clay with no overburden pressure and subject to drying shrinkage would allow a quick undetectable build up of seepage pore pressures and a simultaneous clogging of the filter would be the sort of inversion of design hypotheses that would send all existing dams floating down the river; particularly if one considers that the clogging of the filter would result in the development of full hydrostatic pressure and this would be coupled with no gradual indications from the piezometers. At any rate, on being required to analyse an existing job, one cannot analyse as if one were designing a dam from scratch, but must achieve a much better assessment in analysing the change of factor of safety which must be considered as a result of the anticipated changes of stress and possible margins of error.

To cut a long story short, when the dam was filled the best of records were noted for all instruments (piezometers, internal settlement gauges and inclinometers, surface settlements and movements). The dam truly represents a considerable advance, in particular with respect to the outer slopes over and above the cross-sections which Terzaghi initially used. This is so despite a lesser use of selected compacted materials.

In the lower Figure, I have schematically indicated the cross-section that I would use for a hypothetical dam of similar height. Indeed, I have used just such a cross-section recently.

The principal points with regard to this cross-section that should be noted are in particular the fact that the first material to be used would be rockfill and this would only be used for the first stage upstream and downstream cofferdams, for a higher downstream rockfill toe where the prime stability problem is encountered and for a slightly widened rip-rap zone (the remaining upstream surface is covered with rock spalls of quarry waste). For a rapidly rising and falling flood hydrograph the rising of the upstream cofferdam can be achieved by using compacted clayey fill.

The major stretch of the chimney filter should be inclined upstream so as to avoid any interference of the flownet pressures on the critical downstream slip circles. In this way there will be no unfavourable change of the factor of safety due to the reservoir filling on the important downstream zone.

However, the base of the dam is the heavily loaded central area and should be retained as an impervious contact. This is a significant item in most foundations, and especially in saprolites and partly weathered jointed rock. In such materials the permeabilities are often decreased to one-tenth to one-hundredth by the pressure which tends to tighten the joints. Thus a clay blanket upstream is very inefficient in comparison to a clay blanket under the central portion of the dam where the contact area which would normally be of the order of 2 H can be made equivalent to 20 H or as much as 200 H in as far as concerns the possible seepage losses through the upper horizon of the foundation. Moreover, both by shortening the drainage path and by means of the decreased flow, the problems of the hydraulic head on the downstream drainage blanket are considerably reduced, thus dispensing with a thicker sandwich drain and so on. Such uplift heads have to be minimized by the drainage blanket because considerations of stability on the final downstream stretch are influenced by the pressure acting on such possible downstream slip circles.

Although such a section has not been used before, it embodies the accumulated interpretation and all the precedents of our experience. It can be seen that even in a dam two and a half times as high as the Terzaghi dam, both the upstream and the downstream slopes have been steepened considerably and treatment for the control of seepage and uplift have been minimized. On the upstream slope a very low factor of safety can be accepted for the extreme hypothetical rapid drawdown case, primarily because of the very low level of consequence of failure, and because of the very low degree of reality of the routine tests and the analytical assumptions. On the downstream slope the design principle of avoiding a change of condition due to the filling of the reservoir establishes a conscious benefit of a "pretest" principle which is widely employed in Civil Engineering. In particular, in dam engineering it is the very sound principle upon which the concrete face rockfill dam has accepted downstream slopes of 1:1,4 and this is close to the angle of repose of the soil. This is also unanimously accepted as the most stable dam despite such a slope. Under such a "pretest" design principle it is desirable to force somwhat higher end-of-construction pore pressures and consequently to accept lower end-of-construction factors of safety, so that if at all possible, the change in the factor of safety due to the filling of the reservoir may be positive, or at the worst, be minimal or negative. This is really due to the dissipation of the construction pore pressures in spite of the filling of the reservoir. Such a design concept has the backing of the precedents which were satisfactory for the classic hydraulic fill dams which have a period of critical stability during construction, but which, after construction, are stable.

As recently emphasized (cf. V.F.B. de Mello, General Report, "Philosophy of Statistics and Probability Applied to Soil Engineering", 2nd Intern Conf. on Statistics and Probability Applied to Civil Engineering, Aachen, 1975) the very helpful principle of extracting a design advantage and an assurance from the Bayesian prior probability decision theory must be viewed much more soundly and profitably in the light of an assessment of a specific case, in particular as it progresses from one state of stress-strain-time conditions to the next.

With respect to the problem of the foundations of earth dams on the abutments much experience has been collected about two very frequent phenomena. These are the compressible "porous red clay" on the one hand, and the saprolites which frequently underlie the dam on the other.

To present a minimum overall perspective of the experience accumulated and interpreted would require another lengthy paper.

As regards porous clays, Figure 12 merely purports to summarize the data on the problem which was considered of greatest concern in the Três Marias Dam. The porous red clays were computed to give an estimated settlement of up to 2,0m. The measured settlements were about 70% of the computed values. In the oedometer tests with and without soaking, it was demonstrated that the collapse settlement would be insignificant under the central zone which, due to a relatively high embankment loading, would have suffered signifi-

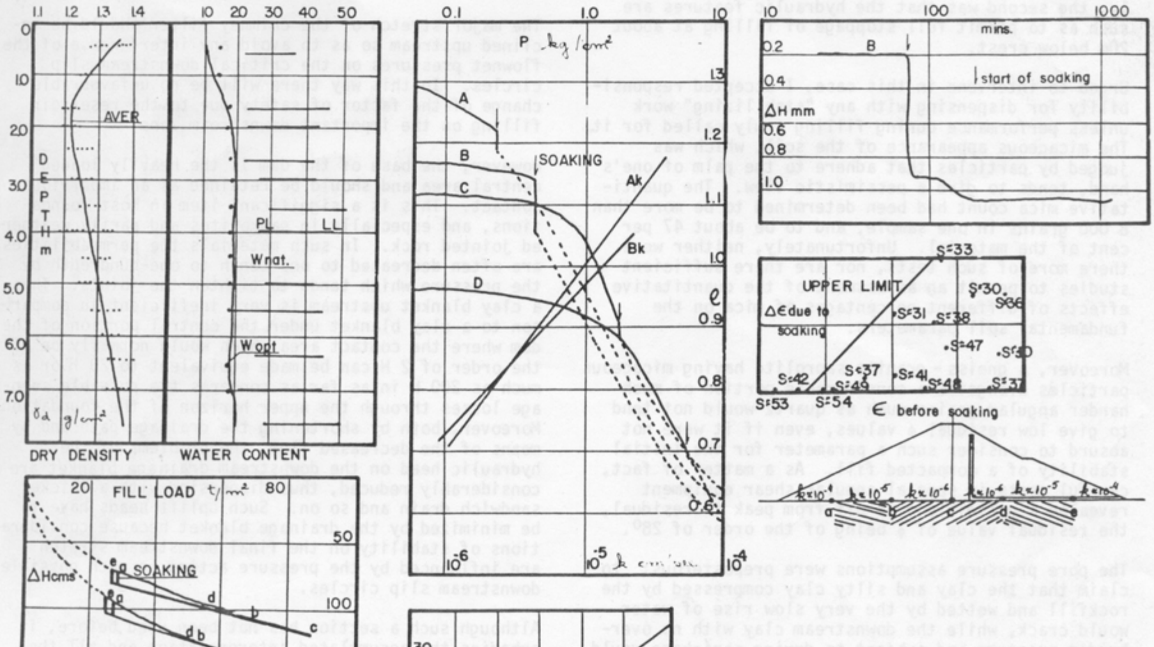


FIG 12 FOUNDATION ON POROUS RED CLAYS

cant compression. Obviously the similarity with the problem of losses is great, but with the important exception that since our soils are produced by lixiviations by rain infiltration over thousands of years, it is impossible to collapse them merely by watering or submerging; it is necessary to apply stresses above the nominal preconsolidation pressure and then to soak them.

The foundation settlements along the centre line were computed at several sections, and the specific differential settlements were considered acceptable because they established a predominantly dish-shaped concave upwards form with values generally less than 1:200. (Presently our criteria are more concerned about changes of inclination, especially if they concave downwards). It must be remembered that in a homogeneous dam section it is easy for there to be a sufficient width of dam section if the dam is to be immune to cracking. This is so particularly if one considers the variation in the transverse direction and the instantaneous settlement. The width of section that would be satisfactory for the core of an earth-rock dam would be about 0,3 times the height. In short, the settlement and the cracking problem were handled, although minor accomodations were made so as not to affect the design or the performance of the Tres Marias Dam. The foundation permeability also proved not to be a problem. A very interesting point is the favourable change of permeabilities that are created by the significant compression of the soil. The permeability should tend to decrease very considerably under the centre, particularly in comparison to the change under the toes. This constitutes or causes a gradient of permeabilities beneath the dam. Unfortunately, no direct or significant data have been collected to date from piezometers which show the consequences of such changes of permeability on the flownets across and under the foundations. It must be emphatically pointed out that such benefits do not favour the use of shallow sections for the dam on the abutments.

The important problem of the loss of strength of the porous foundations when they are soaked was not given any special attention. In fact, it was not considered in the prototype or in the study of its behaviour, principally because the foundation flownet did not establish that the pressures were of a sufficient magnitude, partly because of the underlying more pervious layers, and partly because the topography favoured transverse and longitudinal drains, and finally, doubtless, also partly because of the presumed favourable permeability gradient which has been discussed above.

In the case of saprolite foundations it may be stated as a summary that the observed settlements have been very small in comparison to those that would be computed on the basis of oedometer test results if one used the softer pockets of material, and these settlements have been essentially instantaneous. A negligible development of pore pressures with contruction has been observed even below the ground water level. The results appear to concur with a theory of stress distributions for equivalent deformations (cf. V.F.B. de Mello, "Thoughts on soil engineering applicable to residual soils", 3rd Southeast Asian Conf. SMFE, Hong Kong, 1972). For instance, at the Volta Grande Dam at a section for which settlements were computed to be between 20cm and 110cm (for the stiffest and the softest oedometer curves), measured settlements were actually 23cm. If one has regard to the permeability the average effect of the compression is similar to that mentioned above for porous clays and appears to be well confirmed, although erratic highly preferential minor flows along open cracks do persist. Finally, once again, in the area of shear strengths and shearing deformations true evidence is lacking, but the behaviour appears to be much better than would be expected from a conventional soil mechanics approach, possibly because of the principle that it is the stiffer nuclei that carry the major part of the stresses.

In summary and to conclude, it has been the intention of this presentation to portray some of the principal developments in the design and construction of the predominantly compacted earth sections for dams in Brazil which have been constructed over the past score of years.

Although most of the dams involved in-river gravity concrete sections and wrap-around earth-rock sections so that a step by step development has been possible and important, and although it may well be affirmed that the true problems of dam engineering lie in details which are used at such sections and at the transitions between them (details that are not broached in academic circles), in view of the dominant interest at this conference which is Soil Mechanics, the presentation has been concentrated towards the purely earth sections.

Regarding the simplest case of the in-river embankment designed on sound rock, the experience herein summarized has been dominantly conditioned by the following:

(a) Meteorology, hydrology and water use.

The periods of very low flow, moderately low diversion cofferdams for dry-period risk.
Long dry periods, winter months April to November, permitting a rapid rise of the partial upstream zone of the dam for the flood-period cofferdam.
Very high flood flows which required high spillway gates that permitted a controlled filling at the higher elevations, if this was at all desired.
A proportionately small drawdown during the earlier years of operation.

(b) Foundations.

The principal problem of the foundation is its permeability in the upper horizons, as well as the permeability of the core-contact discontinuity. This is a difficult factor to assess, the degree of confidence attached to the investigations is low, and there are no significant methods available for deriving a benefit from the desirable "pretest principles".

(c) Materials.

Generally sand and gravels not available.
Vast and cheap availability of clayey soils, unsaturated, at close to the Proctor optimum.
Both residual soils and saprolites were used with relative ease and success and moderate adjustments only were required, and these were a function of the wide variety of available earth-moving and compaction equipment.

If one has regard to the design principles and criteria which operate in the area strictly reserved for Soil Mechanics, the experience which I have attempted to summarize in this paper, in particular

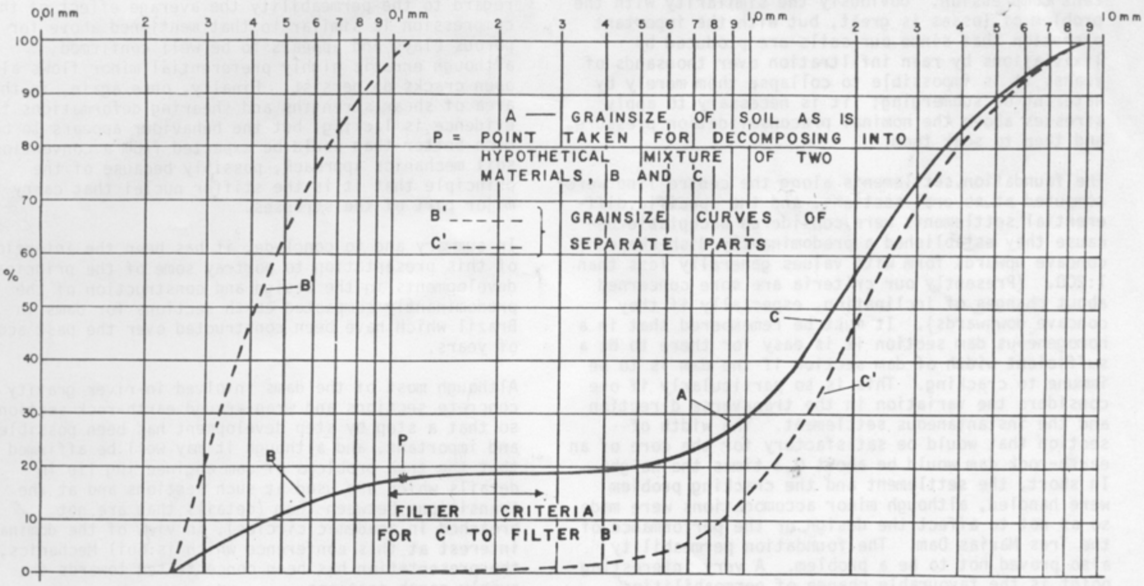


FIG. 13 SIMPLE TEST ON GRAINSIZE CURVE TO CHECK UNACCEPTABLE SKIP-GRADING W.R.T. INTERNAL EROSION

with reference to the simple case of the homogeneously compacted earth-dam section, has indicated or demonstrated that the simple academic cross-section has been thoroughly revised under the Engineer's basic mandates, and all this serves to exemplify few important lessons.

(a) Challenge "precedents" routinely implied as the geometry of the cross-section, and as fallacious intuitions based on Soil Mechanics index parameters.

(b) Be ready and eager to let Nature teach from her performance and revise theory to fit the facts.

(c) Consider the problem from the general to the particular and avoid a premature compartmentalization of "specialities".

(d) Once the conditioning problem has been indentified, distinguish between the scientific preoccupation which leads one to delve further and further into the problem, and the Engineering attitude of "design for positive control", and thus transcend the problem by a dominant course of action.

(e) In as far as possible, invite the dominant problems to occur during construction, both in order that one may benefit from the "pretest principle", and to minimize the risk when the reservoir takes control, and when any damage would be immeasurably greater. Strictly within Soil Mechanics, the desire might well be to force as low as possible a construction period factor of safety such that the change in the factor of safety due to the reservoir filling might be small. The means at the disposal of the design engineer are compaction specification with regard to the air pores and the construction pore pressures, and drainage features which control the operational pore pressures.

As a closing message, I should emphasize the importance of distinguishing between the real and the serious problems, and those that are secondary or fictitious. There should never be any concern about a hypothetical future or a present case of rare coincidence, which embodies a lack of problems. Man will always find problems enough to satisfy his need for "play", since in a deep sense Work is Play. The importance of discerning the significant problems concerns us deeply. On the one hand, Man's capacity to see, face and solve problems is not infinite, but quite definitely finite; so, to the extent to which we devote ourselves to non-problems, we run the serious risk of letting the real problems overtake us. On the other hand, we professionals of the so-called developing countries have a tremendous responsibility to our society to avoid the pitfalls of Modernization prior to Development. Let us not create prematurely in our engineering works the more sophisticated modern needs, in the light of second-order problems, before we develop the means to tend to them, or the right to play with them. Each Society has the right and obligation to set, in Decision, its own level of requirements. If we begin to impose, prematurely, on developing countries the second-order levels of requirements of the highly modernized societies which seek for new problems, we may be doing no more than burdening the project with higher costs.

Development first and Modernization immediately subsequent, suggests to me a convergent infinite series that leads to a solution, and satisfaction. But Modernization prior to Development is surely a divergent series that can only lead to frustration, to an ever-widening gap.