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Considerable attention has been given in recent years to the problems of transverse cracking in dams and associated phenomena of hydraulic fracturing and piping, and design and construction features in defence against such hazard. Originally the problems were principally associated with differential settlements due to foundations, and rapid changes longitudinally along steep abutments were tabooed. Soon it was also recognized that differential deformabilities within the dam " superstructure " itself can be as important or more, insafar as within the body of the dam any elevation is, to all effects and purposes, a foundation plane to the overlying part of the selfsame embankment. Similarly, stringent requirements have been imposed on transverse slopes within the body of the dam when built in separate phases because of river diversion schemes; since any such transverse first phase slope acts as an abutment to the second phase, the analogies in reasoning are obvious ; it must be remarked, however, as quite unexplainable that the contact of the embankment against the rigid concrete " abutment " of a gravity section has been forced to cede to comparatively less rigorous impositions, apparently in part because it straddles the area intermediate between two specializations, and in part because forcing increases in concrete volumes is a rather expensive proposition.

The problem has been and is being tackled along many alleys simultaneously. Geologic and hydrologic conditions in many areas in Brazil have been observed to accumulate factors in a direction such as to focus on the problem as especially important. The Writer profits of the occasion to summarize his appraisal of the promises and problems connected with some of these alleys, basing his comments on the work being done on many big projects with which he is associated as consultant. For simplicity the discussion will be restricted to the case of a zoned dam on a firm foundation.

The following is an attempt to systematize the trends of thinking and work that have arisen in the wake of the recognition of the problems as due to the oft-times lack of compatibility of stress-strain behaviour of the distinct zones. Figure 1 presents a schematic cross-section of the Salto Ozorio Dam, used herein for easy reference.

1. Firstly there is the problem of compressions of the compacted earth core and the rock shells under self-weight and reservoir loading.

The highly deformable dumped rockfills of up to 15 years ago were substituted by intensely (sometimes excessively) compacted rock shells : because of slope stability it must be recognized as unlikely that the trend towards heavy compaction of the shells will be even partly attenuated.

Under the circumstances, as regards the impervious core two opposite trends have found justification. One, towards use of the soft plastic cores, such as the puddled clay core, on which the Writer has no comment (because of lack of personal experience on the solution) excepting the earnest appeal that quantitative data on design, construction, and behaviour parameters of such cores be investigated, interpreted and published, in benefit of a solution that appears to have high promise if well enough documented. The other, towards added emphasis on the heavier and more careful compaction of the clay core. Heavy compaction of huge volumes of clayey materials has been a routine in Brazil's big hydro projects, and recently the problems have been accentuated in connection with very clayey " red

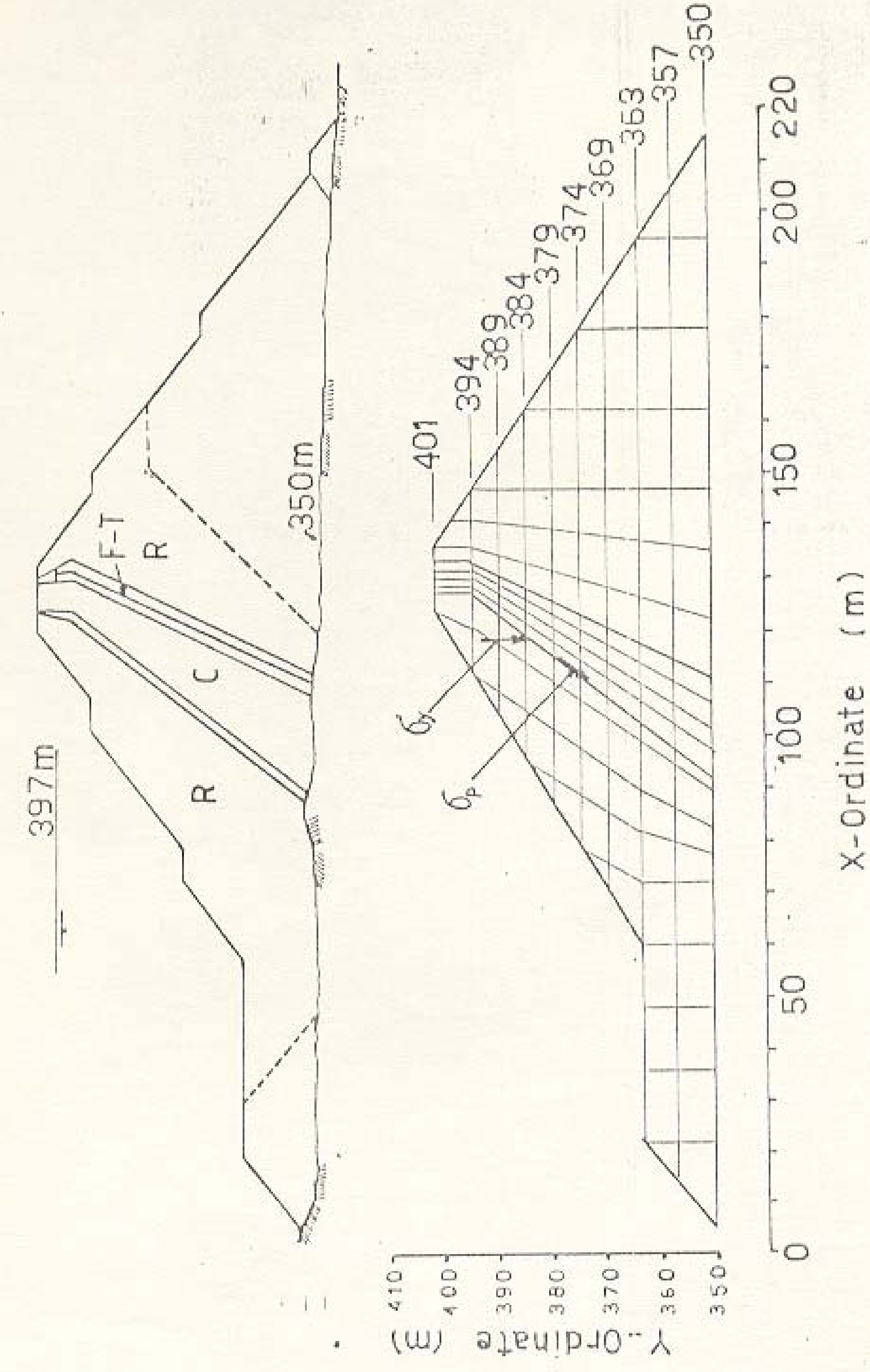

Fig. 1.

soils " derived from basalts. Based on such experience the following comments are submitted in compact statements of first-degree approximation, obviously vulnerable to second-degree reappraisal.

(1) The production trend toward heavier hauling equipment imposes a definite need for even heavier compaction effort, so as to achieve homogeneity despite uneven compression of lifts under haulage tires.

(2) For most clays of Standard Proctor optimum water contents higher than about 30 %, weights and pressures of hauling and compactive equipment have exceeded the desirable range; effects of over-compaction are produced at the least provocation.

(3) Such problems multiply considerably if borrow pits are wet of optimum. Emphasis must be given to as correct as possible direct determinations of $\Delta w = W_{comp.} - W_{opt.}$ by Hilf-type tests without predrying or reuse of sample for more than one compaction test point. Although most soil mechanics test results are subject to comparatively broader dispersions, small changes in Δw may be quite significant towards collateral effects not reflected in routine density inspection testing.

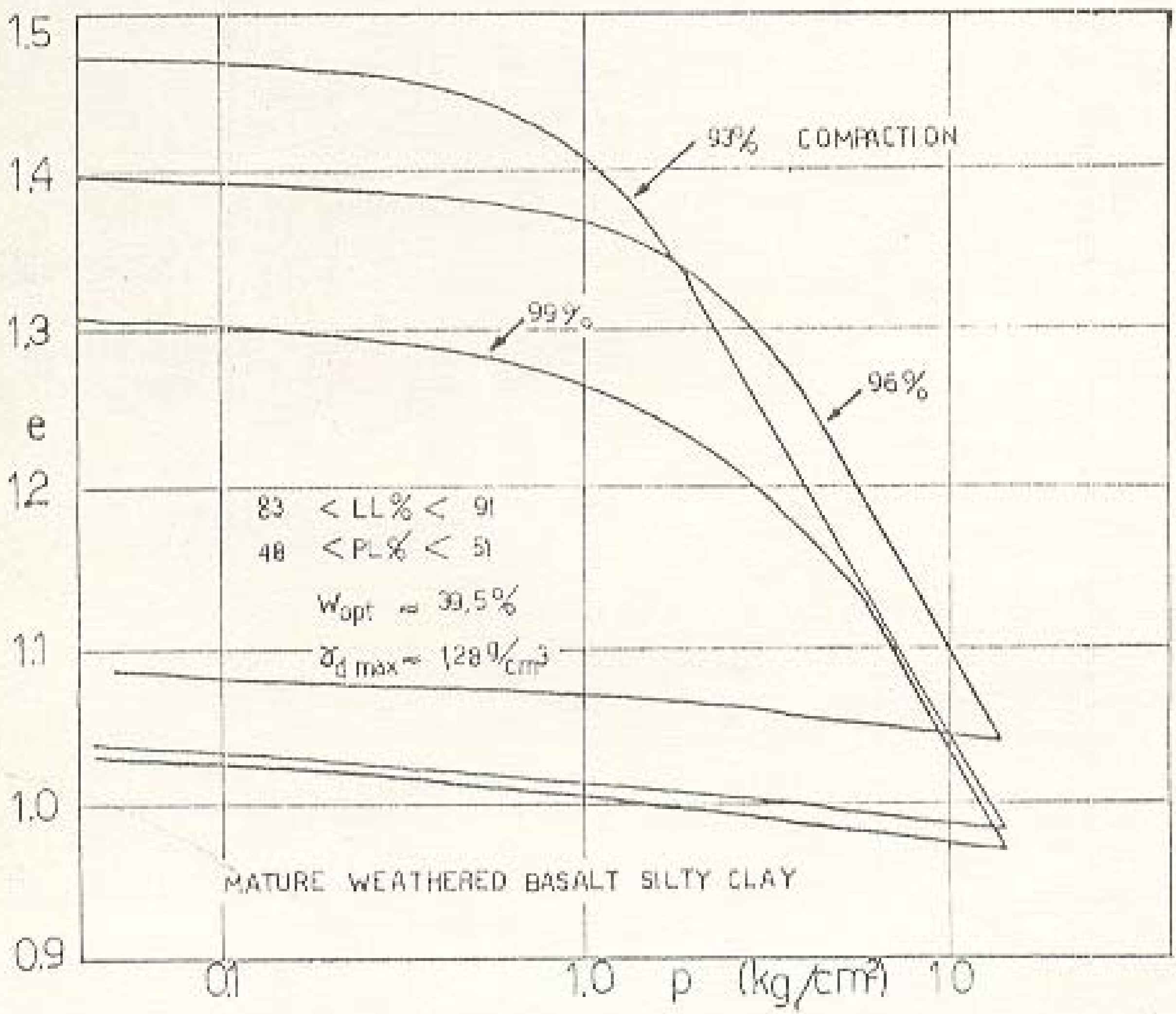

Fig. 2.

(4) Controlling compaction water contents to within limits in accordance with routine specifications does not cover the problem ; the Writer has found it important to determine Percent Saturations in situ in the borrow, and warns against troubles with high saturations ($S \gtrsim 85\,\%$).

(5) As shown in figure 2, very little significant decrease of compressibility is achieved with compaction beyond about 96 % Proctor. Heavier compaction basically achieves a slightly higher " virtual preconsolidation pressure ", with no effect on the compression index. As is more realistically shown in figure 3, the relative incompressibility below the virtual preconsolidation pressure is not significant, especially for higher dams, and especially as regards any possibility of compatibility with rockfill compressibilities.

(6) Definition of the quality of a compacted clay fill through " statistics of averages " by reference to inspection tests, constitutes a satisfactory approach regarding compressibility phenomena, wherein integrations are

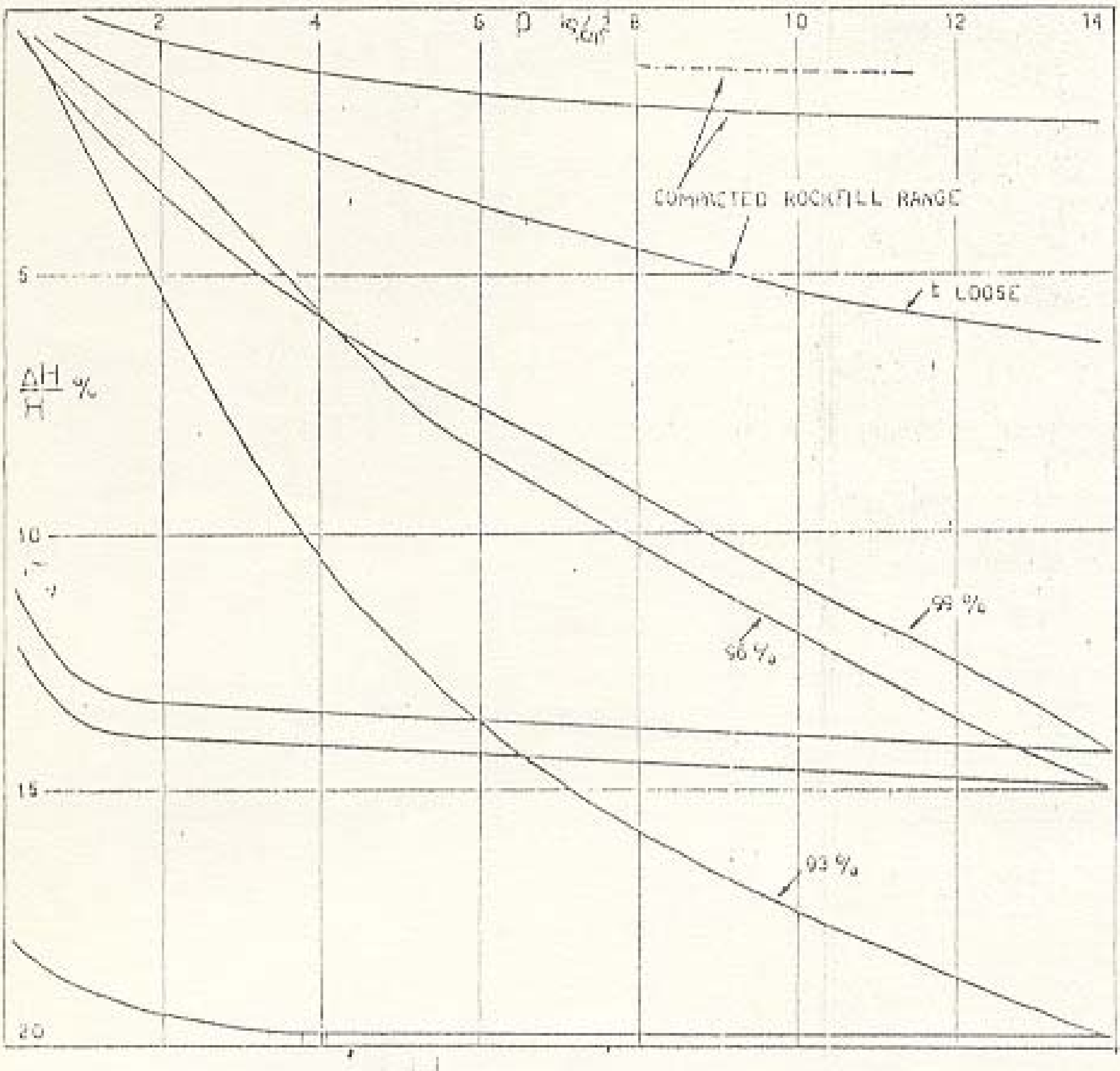

Fig. 3.

involved. In connection with Paper R. 1 the writer would recommend interest in representing the complete Frequency Distribution Curve of percent compaction beyond the established rejection criterion, as shown in the paper " True Representation of the Quality of a Compacted Embankment " V.F.B. de Mello *et al.*, 1959, First Panam, Cosomef, Mexico, Vol. II, p. 657.

2. Secondly, the design of the zoned cross-section has sought to optimize the position of narrow earth cores so as to avoid zones subject to unfavorably low or tensile stresses due to differential compressions. The principal technique used for such design studies has been the finite element analyses, and the principal design solution available has been inclining the narrow zone responsible for the incompatible pressure-settlement behaviour.

Based on the solution developed at the Univ. of California, Berkeley, by Kulhawy, Duncan, and Seed, rapid steps were taken to investigate problems of stress redistributions within higher earth and earth-rock dams. At the Salto Ozorio 53 m dam (1) an early alternative of a central core

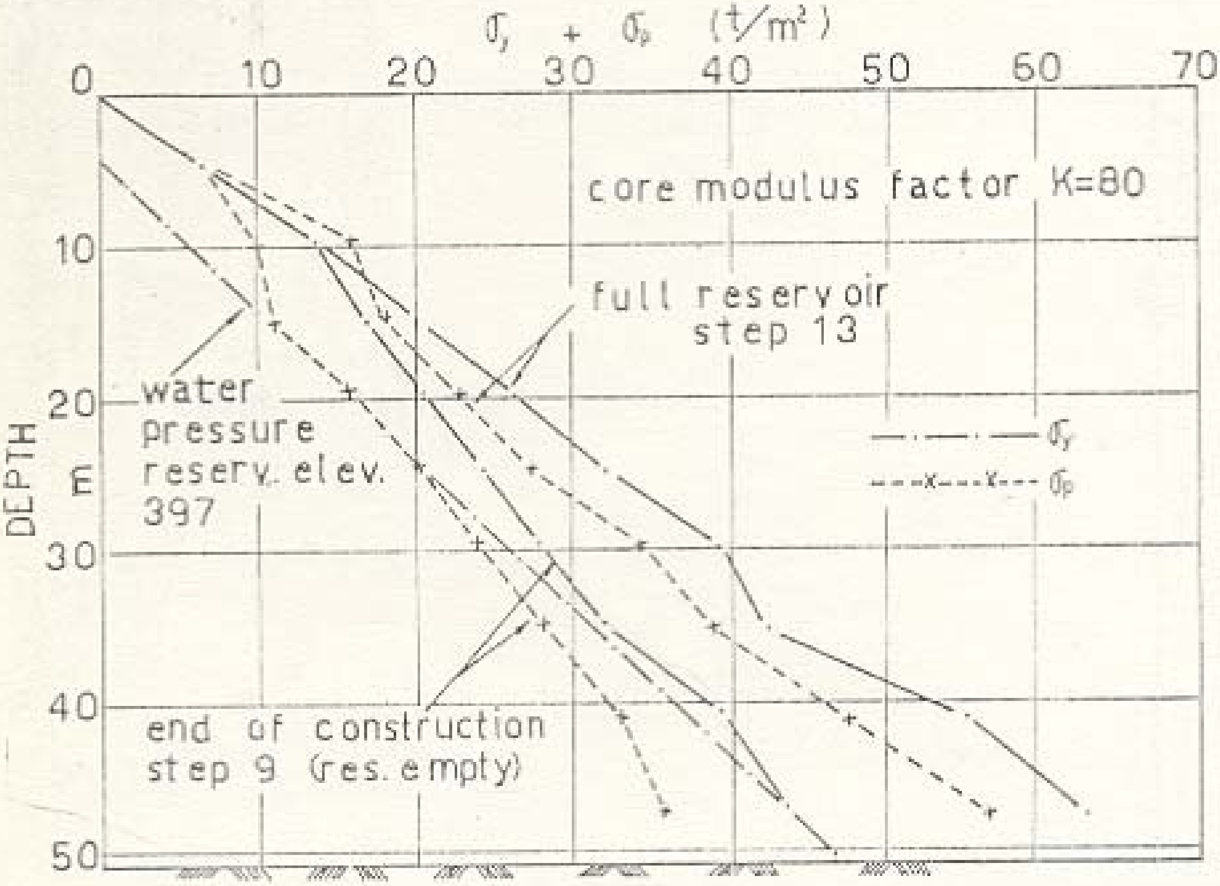

Fig. 4.

(1) The Writer is indebted to Copel-Eletrosul, Owners, Serete-Kaiser Engineers, designers, and to Prof. J. M. Duncan, executor of the finite element analyses.

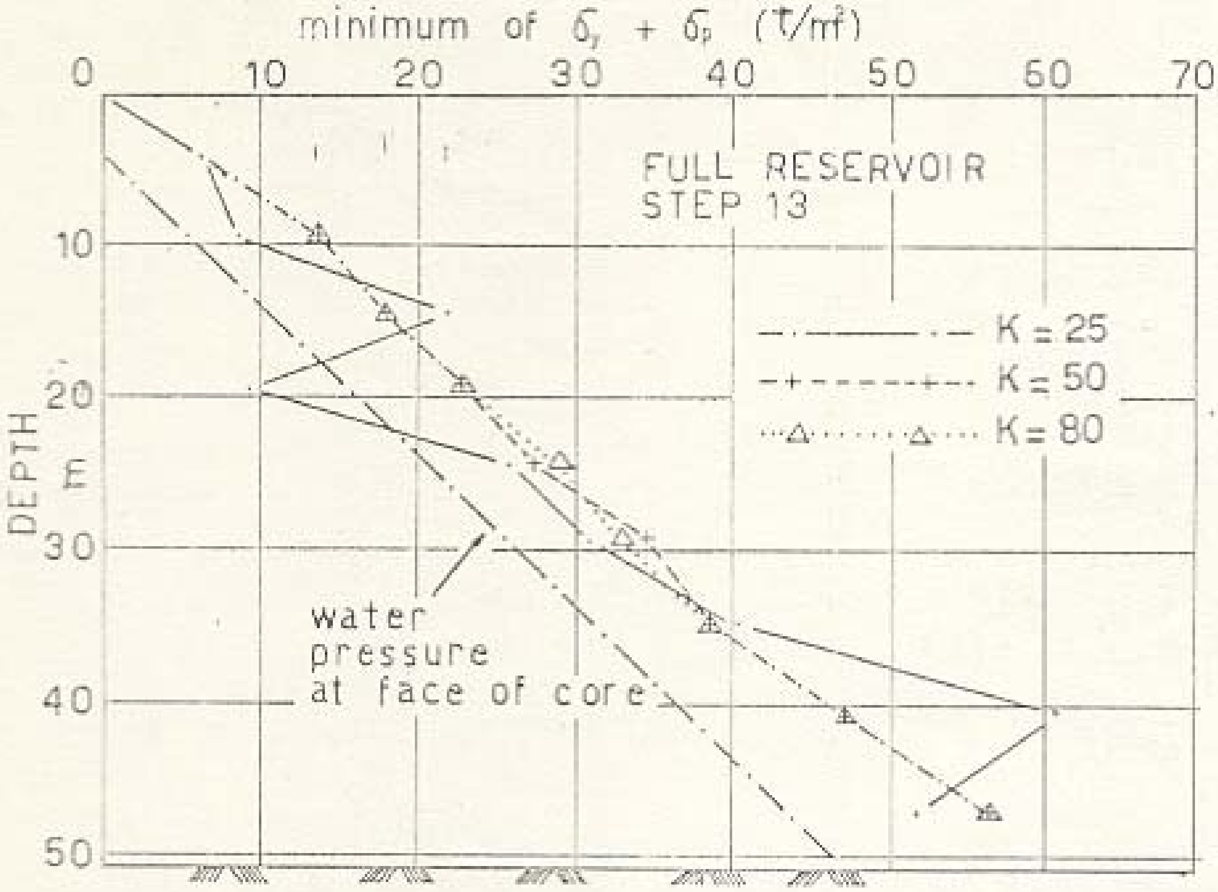

Fig. 5.

was promptly substituted by an inclined core as shown in figure 1 : at the Marimbondo 85 m dam (2) the frequently-vertical chimney drain was sloped over a stretch; at the Itauba 100 m dam entirely analogous to Salto Ozorio, the vertical core of the preliminary design has also been inclined upstream. Figures 4 and 5 present summary indications from the studies on Salto Ozorio.

Many designs have resorted to finite element analyses over the past three years, and papers such as R. 18, R. 26, R. 29 and R. 44 presented to this Conference are good examples, the latter being one specific study towards assessing, through a simplified " single-lift " assumption, the optimized position for a core without increasing fill volumes. In comparison with the potent finite element and finite difference numerical computation techniques the writer would relinquish interest in furthering such simplified analytic studies as explicit or implicit in Papers R. 5 and R. 6.

The results summarized in figures 4 and 5 embodied as the principal difference between the core and shells the Modulus Numbers K of 1,200

(2) Several studies and publications by Decio de Zagottis and Evelyna B. S. Silveira, of Promon Engenharia S.A., Engineers, in connection with this major project of Furnas, Owners, are thanked for the principal introduction of finite element analyses in embankment dams in Brazil.

(rockfill) vs. 80, 50 or 25 (three assumed values for the core) : the construction loading was developed in 9 steps and the rapid reservoir filling in 4 additional steps, the reservoir pressure being applied as a boundary pressure on the upstream face of the core (a valid assumption for a very rapid reservoir filling); the stress-strain characteristics of the shells were handled in terms of effective stresses, while, justifiably under the hypotheses, those of the core were in terms of total stresses; hydraulic fracturing was thus postulated to occur if the reservoir water pressure became greater than the total stress (either vertical, σy, or parallel to the core face σp) at the upstream face of the core.

Further obvious developments are aimed at including, under somewhat slower effects, the seepage forces as body forces computed from the flownet, and the consolidation effects which are of great importance because relative deformabilities will be vastly accentuated : it is with time and consolidation that the clayey core (if wet and more nearly saturated) will prove more compressible, and since meanwhile the rockfill deformations are essentially rapid (in cases where the high point-stress crushing deformations of a secondary type are low), when the time deformations of the clay get to be most significant the ratio of rock Er to core Ec would grow very considerably. In one study (2) it has been proposed to consider consolidation effects through a simulation comprising a drop of the clay Modulus of Elasticity Ec; although the specific case assumed a constant weighted average drop of Ec to α Ec, it is accepted that without too much added difficulty or computational time it will be possible to introduce different values of α as they will vary considerably from point to point within the mesh, and at each point from time to time during finite consolidation increments.

The Writer foresees that as such studies develop, one very important consideration might turn out to be the compression-expansion hysteresis. It is normally assumed either that the nominal " elasticity " moduli in compression and in tension are essentially equivalent, or that the redistribution of stresses does not really imply a time sequence of compression followed by expansion, but rather a mental sequence of superposition of stresses (and consequent strains) as equivalent to " avoidance of a consolidation compression " in lieu of a physical expansion. However, due to time and space effects in dissipation of excess porepressures, it is very likely that certain zones within the core will really be forced to expand and swell after having been compressed. One has been forced to witness, in inspection pits dug into a very well compacted clay core adjacent to a steep concrete gravity face, sizeable zones of material swollen to a muddy medium to soft consistency : the zone is suspected to have been under tensile stresses when the reservoir water reached it. Obviously in such cases a really hydrophilic clay with a Swelling Index C_s not far different from the Compression Index C_c should be much preferred.

In short, finite element analyses are in the process of development with a view to including the changes of stress-strain behaviour that the embankment materials are physically known to undergo under the very stress, deformation, and seepage, changes that the operational history of the dam will bring about. Thereupon, since it can be foreseen that additional computational developments will be comparatively rapid, attention may be shifted to concomitant investigation of appropriate parameters to insert into the analyses.

3. Thirdly, the investigation of " flexibility ", " plasticity ", tensile strain deformability, and tensile strength, of compacted clay cores.

Within this important component of the overall problem the Writer would begin by emphasizing the need to distinguish between laboratory studies and conditions applicable in the field. Hitherto attention has been centered on laboratory investigations on laboratory compacted homogeneous specimens.

In such laboratory investigations there has occurred a rather unfortunate bifurcation within the original and necessary basic aim which was to determine both the deformability under tensile stresses E_t, and the ultimate tensile resistance R_t. Paper R. 10 is an example of an investigation

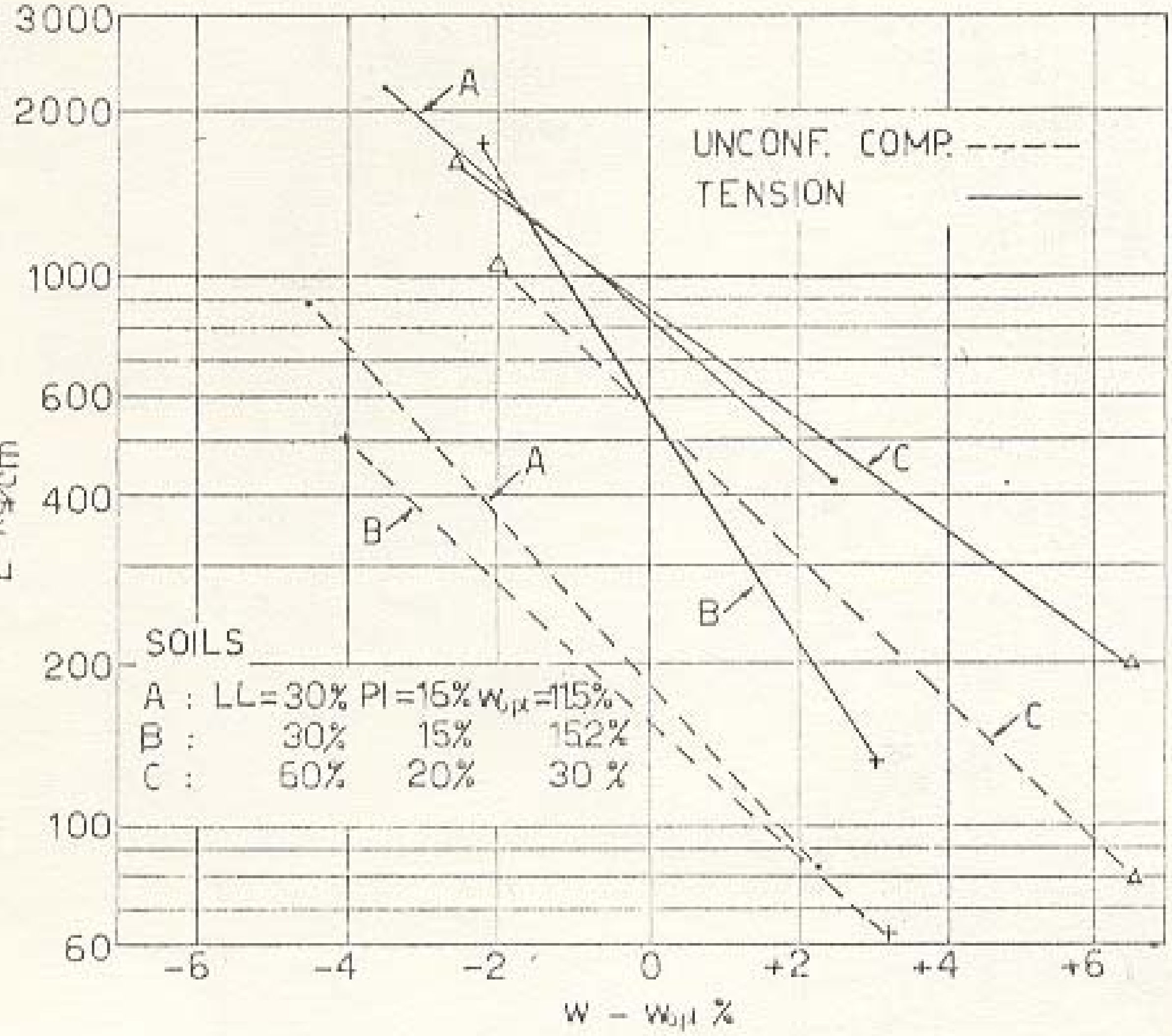

Fig. 6.

exclusively into the latter parameter. At least four different techniques (3) have been employed for comparative tests on R_t, but really a distinct preference should be given to tests such as presented by Bishop and Garga (1969) employing the triaxial equipment, because of the need to determine E_t concomitantly, and to investigate both parameters under different controlled stress histories, drainage conditions, saturation, etc. Figure 6 summarizes some results from the comparisons of compression E_a and tensile E_t moduli, under quick tests on three compacted soils. The results indicating $E_t > E_a$ appear reasonable if one assumes that tensions (capillary) stiffen compacted clays. Such quick tests can be significant with respect to appraisal of susceptibility to cracking under rapid reservoir loading : but the follow-up under slow swelling and saturation can be as fundamental a problem.

A recent study (4) shows that on six soils of $3 < PI < 25$ R_t in drained triaxial tests drops to about 50-66 % in specimens subjected to back-pressure saturation, in comparison with specimens as compacted at Proctor optimum.

There has been much indiscriminate use of terms and concepts connected with the aims at " plastic deformability " to avoid cracking. Much of the pertinent literature automatically associates a higher (unquantified) capacity to deform without cracking, to higher values of the Soil Mechanician's Plasticity Index *PI*. Actually the *PI* is no more than a measure of the range of water contents over which a clayey material exhibits, under atmospheric pressure conditions, the empirically defined " plastic behaviour " ; nothing is thereby established with respect to plastified behaviour under high total stresses, although there is some reason to presume that more clayey materials will lead to higher pore pressures and thereby to more " plastic " stress-strain behaviour in compression. However, under the same token might it not be that under a stress reversal to tensions, the more clayey material may exhibit higher negative pore pressures and brittleness?

At any rate, the principal point is that it is meaningless to discuss the " potential plasticity " of a type of soil, when what really matters is its plasticity as compacted. A simple first-order reasoning has prompted the writer, over the past dozen years, to plot Optimum Water Contents Wopt vs. Plastic Limits *PL*, on all investigations of clayey borrow pits. Since specifications automatically impose compaction at close to the Proctor optimum, one automatically verifies if the soil as compacted will lie above or below the Plastic Limit. Figure 7 has been drawn up to collect some of such results derived from sundry clayey residual soils (principally from granito-gneisses, sandstones, quartz-schists, and basalts). The results may appear surprising, and may refer only to residual soils : but the important

(3) " Comparative analysis of some tension tests in compacted soils ", N. Gaioto of Promon, D. Sc. Thesis at Univ. S. Paulo, 1972.

(4) " Notes on the tensile strength of some compacted soils ", P. Cruz and G. Mellios, 1972 Seminar of the Brazilian Com. on Large Dams.

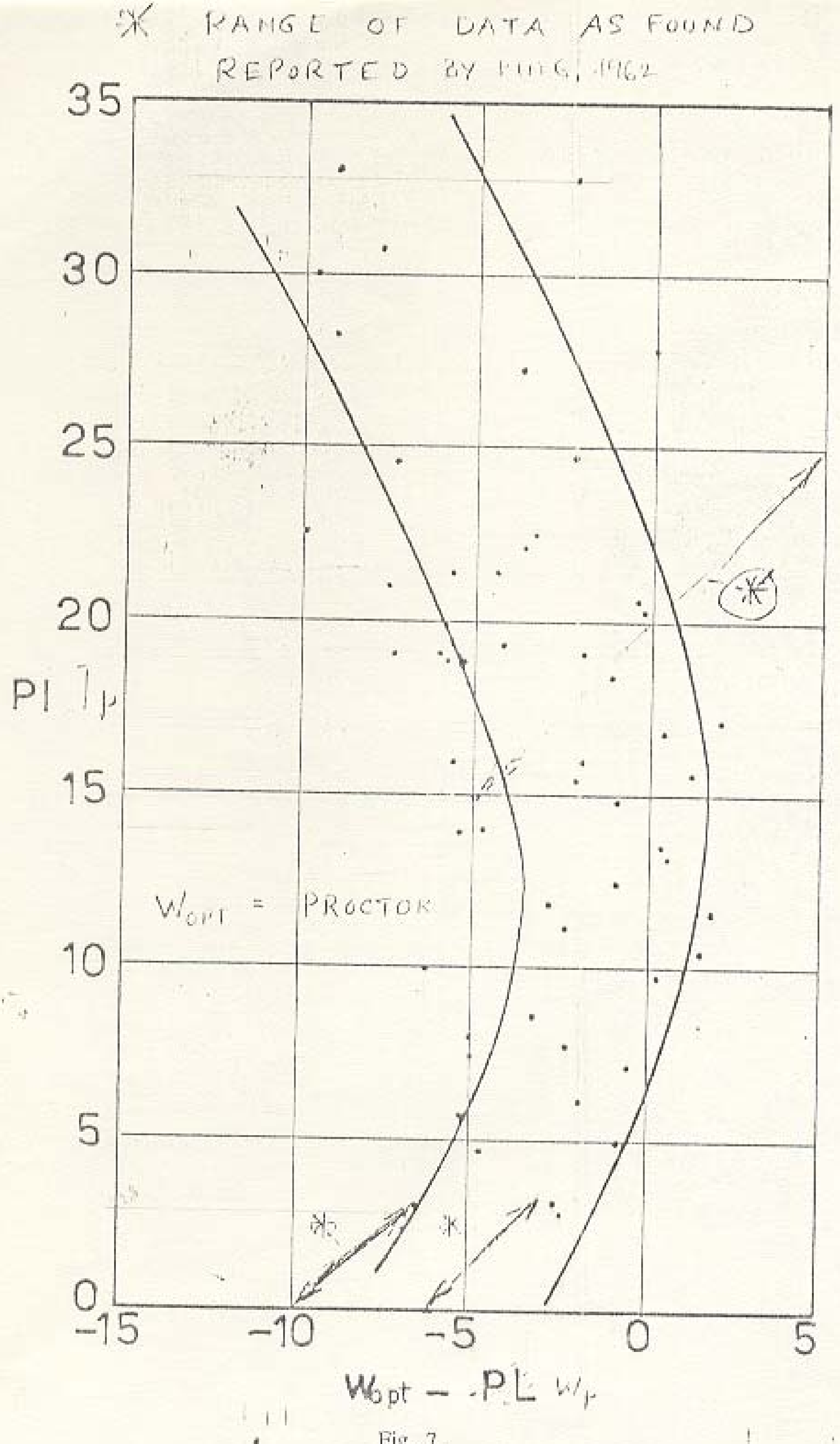
* RANGE OF DATA AS FOUND
REPORTED BY
35
30
25
20
15
10
5
0
PI
-15
-10
-5
0
5
W_{opt} = PROCTOR
Wopt - P.L

Fig. 7.

conclusion would be that only with soils of $5 \gtrsim PI \gtrsim 22$ does one have reasonable probabilities of compacting the clayey material at water contents higher than the plastic limit. The fact that residual clays of high *PI* values may be compacted much dryer than the Plastic Limit may merit special concern, because of the presumption of concomitant inferred brittleness.

In line with these thoughts the Writer has two side comments. Firstly, there would be considerable interest in more quantitative data as to the real benefits sought and achieved by the admixture of bentonite into the core material as mentioned in paper R 6. Secondly, it may be reminded that in the Writer's opinion (5) the Atterberg Limit tests for plasticity are really an unfair measure in most residual soils. Clayey saprolites may exhibit a very high " plasticity potentiality " (upon full plastification by remoulding) as reflected in those routine identification tests : but under in situ or compacted field conditions such soils generally do not behave plastically. For instance, in the oedometer curves of figure 2, " plastified " clays of similar *PI* values would exhibit C_s values well in the range of 0.15 to 0.3 C_c whereas the residual clays reported swell so little as to exhibit values $C_s \simeq 0.05\ C_c$.

Despite the obvious interest in such laboratory studies of parametric variations, indispensable towards furnishing a feel for the core behaviour, the Writer would guard with prudence against relying on tensile deformability and tensile strength as a design expedient. It appears that design studies should aim at avoiding any tensile stress whatsoever, and that the abilities to withstand deformations without cracking should be sought with interest as reserve defences. The principal reason why such a stand must be recommended lies in the questions of field compaction and compaction controls.

Firstly it must be emphasized that in the Writer's experience clays of Wopt $\lesssim 30\ \%$ require careful field investigation and decisions on construction equipment and specifications in order to achieve a truly massive, cohesive " fabric " of the compacted material. Very frequently at the dryer water contents required to achieve (with present-day equipment) high percent compactions the fabric is of clayey nuclei pressed together but not plastically stuck together : in such materials frequently the permeability tests run on undisturbed block samples yield 100 to 1,000 times higher coefficients than as obtain in laboratory compacted homogeneous specimens, and if the test is run with water appropriately dyed and after some percolation the specimen is pulled apart in tension, it is found that all the flow was preferential along the contact surfaces between the clay nuclei. On the other hand, at wetter than optimum conditions there is an immediate tendency toward formation of over-compaction laminations.

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(5) V. F. B. de Mello, " Thoughts on soil engineering applicable to residual soils ", Southeast Asian Cosomef, Hong-Kong, 1972.

Obviously as a first step it is indispensable that testing should be on field compacted undisturbed block specimens. The Writer would recommend, with respect to tensile fracture susceptibility, the double radial permeability tests in cylindrical specimens with a concentric hole, such as used in Rock Mechanics, with seepage causing compression in one instance and tension in the next.

However, the principal problem concerns the nature of construction and inspection operations, and the fact that whereas the statistics of averages adequately covers the compressive strain phenomena, it may well be a single plane of discontinuity that may absorb a large proportion of tensile strains calculated as pertaining to the postulated homogeneous mass around it. Such thoughts justify the basic premonitions against acceptance of a design computation based on tensile stresses and strengths.

4. Fourthly, for the sake of completeness, mention must be made of the studies on erodibility and wash-through, and recommendations on " self-healing " filter-transition materials.

In very valid engineering reasoning, one of the measures of prudence in design is to check on the defences available in case the problem of cracking does develop. On the one hand there is the question of clayey core resistance to internal erosion, and to washing-through the interstices of the filter : there have been several studies on these problems in former Conferences, but the subject was not brought up at this Congress. The Writer would again emphasize that significant tests for such " trigger-reaction progressive phenomena " must be based on undisturbed block specimens from the core-filter interface as compacted with all the local irregularities that have not been reproduced in laboratory-compacted specimens.

On the other hand, it is the Writer's impression that there is no published attempt at quantification of the material and behavioral properties associated as desirable in a " self-healing " filter-transition material. Considerable reference to the subject may be found in Project Reports, but apparently no attempt has yet been made to systematize and justify the intuitions associated with the recommendations, and no laboratory research has been reported with respect to checking on such preliminary indications.

This subtopic will probably justify considerable interest for future study and discussion.