III SEMINARIO COLOMBIANO DE GEOTECNIA
Memorias

PROBLEMS AND SOLUTIONS FOR EMBANKMENT DAM FOUNDATIONS ON WEATHERED SOIL HORIZONS AND CRACKED ROCKS

Por: Victor F. B. de Mello,
Soil Engineering Consultant

Bogotá, Agosto 6-10 de 1.984

PROBLEMS AND SOLUTIONS FOR EMBANKMENT DAM FOUNDATIONS ON WEATHERED SOIL HORIZONS AND CRACKED ROCKS

Victor F.B. de Mello,
Soil Engineering Consultant

1. It is obvious that one cannot generalize on weathered soil horizons except in a highly simplified and idealized condition. Each weathered horizon must, by definition, be associated with the macrostructure (joints, lenses, bands, etc..) of the respective parent rock, and also with the respective microstructure defined by its petrography, mineralogical composition and so on. But, there can be some parameters that are common to many of the tropically weathered, unsaturated, horizons of residual soils and saprolites. Recognizedly we must begin by elucidating under what definitions the two terms are being used in this paper: the saprolite disintegrates moderately (under light to heavy mechanical handling) into a soil, and behaves in all routine soil tests as a soil, but is characterized by exhibiting some preferential in situ behaviors (permeability, tensile and shear strength) along relict planes of the parent rock discontinuities (joints, stratifications,schistosities); meanwhile, the residual soil would be so maturely weathered a saprolite, that even the dominances of joint planes have been effaced, so that the soil behaves essentially as an isotropic homogeneous geostatic mass.

In some respects it is also far from possible to generalize on cracked rocks. The intent is merely to generalize on rocks that present, near the transition of the bedrock to weathered rock, a characteristic horizon of open cracks, generally by the physical actions of stress release and differential thermal effects that result in tensile cracking. Such cracks have been found to be very common in gneisses and granites and dense basalts, as well as in several of the sounder, more brittle metamorphics. The intention is to draw from such case histories some lessons of generalizable reasonings applicable to other analogous situations.

For the foundations of embankment dams one must generally consider principally problems of shear strength as affecting bearing capacity and slope stability, those of settlements, those of permeability, and those of possible retrogressive internal erosions of piping. Obviously in the case of dams on cracked rocks the only questions to consider will be those of permeability and of eventual piping. In the case of foundations on saprolite horizons I shall propose that in general we may without difficulty set aside the problem of bearing capacity or slope instability, and therefore will be able to concentrate directly on the problems of settlements, permeability, and possible piping. The corresponding treatments will be discussed.

2. Earth-rock dams on saprolites and weathered rocks. Supporting quality requirements of the foundation horizon.

2.1. Shell foundations, requirements for rockfill.

Bearing capacity and slope stability problems only affect the horizons supporting the shells, and principally the onter parts of such supports (e.g. roughly beyond 45^0 lines descending from the crest). Assuming that a compacted sound angular rockfill slope may well be of the order of 1V:1.3H (i.e. $37,5^0$), the question of an unsatisfactory supporting horizon would only arise if under the construction-period incremental loading conditions, the net shear strength of the supporting horizon (up to the stress level under consideration) were smaller than the necessary $37,5^0$ line. This can be set aside very easily, firstly by excavating somewhat deeper wherever necessary, since we generally find that in saprolite horizons the strength increases with depth. There is a simple dictum in dam engineering, that the adequate supporting layer must be sought as functionally providing for equal-to or better-than the immediately overlying zone of the designed embankment. At zero to modest stress levels, within which a saprolite seldom fails to exhibit some cohesion, it is not difficult to reach depths guaranteeing the strength conditions as indicated.

Using the SPT index, since a well-compacted clayey soil (PC $\geqslant$ 98% Standard or Normal Proctor NP), such as will not have any construction-period instability (not wetter than 1.05% w_{opt}, with degree of Saturation $S \leqslant$ 92 to 95%), will generally indicate SPT $\geqslant$ 15, it can be profitably emphasized that the in situ long-term strength improvements are such that an SPT of about 12 in a residual soil often reasonably matches the SPT $\geqslant$ 15 on the recently compacted embankment made of the same soil. Note however that such an indication is strictly approximate since the SPT is often quite distinct from one company and country to the next, and it is very worthwhile running some check tests in order to calibrate one's index. For purposes of shallow inspection testing to confirm an adequate bearing surface during construction, the Percent Compaction PC% index may be used in preference, and thereupon materials in situ with PC $\geqslant$ 93% (approx.) may well prove better than a recently compacted clayey layer with PC $\geqslant$ 98%. The important confirmatory indication to be secured is that the saprolite in situ is not saturated and highly impervious, so that it can be asserted that the construction-period loading does occur under drained conditions, the fill seldom rising more than a few layers (e.g. 4, or about 0.6m) per day. Construction-period pore pressures in saprolite foundations have most frequently given very low values.

It may be asked why, in general, it may be sufficient to achieve adequate construction-period drained stability of the supporting horizon. The principle is that if under incremental loading we guarantee a $\phi' \geqslant 37,5^0$(the external slope), then for the upstream side the reservoir filling and corresponding introduction of u corresponds to moving into a strongly hysteretic stress trajectory, of effective stress σ' decrease, significant OCR (overconsolidation ratio) and consequent increased strength at any given σ' in comparison with the loading condition. In a well-designed dam cross-section, for the saprolite under the Downstream DS zone the flownet pore pressures should also correspond to introducing a certain OCR and consequent improved strength for a given presumed σ'. In both cases, the stability analyses will be assured of working under pretested conditions (during construction), and therefore under assurance of Factors of Guarantee FG of the nominal stability analyses, therefore considerably safer than under the FS conditions of the construction-period loading.

Recent confirmatory evidence is shown, Figs.1 and 2 , to the effect that the open excavation level reached by the use of a D8 tractor or an equivalent loader, meets the requirements of adequate support for rockfill shells at about 1V:1.3H ($37,5^{o}$). Such general indications can and should be systematically checked in different saprolite horizons, so that prompt and easy indicative construction specifications can be adjusted for each project and jobsite. Very special caution must be exercised in saprolites that exhibit strong anisotropy (schistose weathered rocks) that is not revealed by simple SPT and PC% indices.

2.2. Core foundations.

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

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

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

Well, one of the most undeniable experiences in saprolite foundations is that the decision to support the core on a firm foundation excavated mechanically to definite geometric grade is not only acceptable, but far preferable, technically and economically, to the attempt to excavate down to a "sound rock surface" (cf. Fig. 4a). It is characteristic and inevitable that the sound rock surface will be found in a most irregular topography (including reentrances and negative slopes etc..): this causes great difficulties, technical and logistic (therefore economic) to achieve the necessary perfection of cleanup (air-jet) and compacted backfill. Note that very frequently it has been preferred to use generous applications of dental concrete and even a thin layer of minimum concrete filling over the entire surface, deeper points receiving greater thicknesses.

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

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

3. Recommended internal impervious blanket.

It is general experience that saprolite foundations tend to be more pervious than the compacted embankments: so also generally a top horizon of rock more intensely open-jointed would suggest the need for some sealing treatments. As was emphasized in my Rankine Lecture 1977 (DP-1) the priority treatment would be of physical exclusion of extreme situations, and therefore, of grouting the wider open joints: this concept, although embodying a priority design decision, will be detailed in items 4 and 5. For the present I restrict myself to applying Design Principle 3, DP-3 "Notwithstanding DP-2, entice integration of effects and cumulative cooperation within the said universe". If there is any pervious horizon under the embankment dam, once the validity of behavioral averages (flownet etc.) has been assured, two solutions have been automatically considered: on the one hand to create a cutoff (a vertical "impervious" barrier across the pervious horizon); on the other hand, to increase the seepage path by employing the classical external impervious blanket.

Let us set aside the impervious cutoff element which will be employed whenever practical, but in no way affects the adoption of the internal impervious blanket, which at worst becomes superabundant and therefore could be relaxed in quality. A question often asked is how carefully cleaned and tight must be the contact between internal blanket and underlying horizon. In any such cross-examination one must begin by assuming (and guaranteeing in construction) that the contact sheet (few mms to cms) will, at the limits, have permeability properties between those of the pervious foundation horizon and of the impervious compacted fill: then the answer comes automatic. What difference can it make if a pervious horizon x ms. thick becomes (x + dx) ms., or if the internal blanket y ms. thick becomes (y + dy) ms.? None, obviously (Fig. 4b). The intuitive worry over a contact layer is associated with a thin layer appearing, presumed to be much more pervious than the permeable foundation horizon. Similarly a question asked is, how selected and well compacted must be the internal blanket fill. Once again, assuming that this fill is unquestionably more impervious than the foundation (say 10 to 20 times) the beneficial effect on separating the flownets of embankment and foundation is already inexorable: the question of quality is not crucial, but involves partly the relative settlements and deformations affecting redistributions core--transition.

Presently the use of a core slightly inclined to upstream is favoured because of differential deformations and the desire to minimize core hang-up. At any rate, as was discussed in the Rankine Lecture, a chimney-filter somewhat US-inclined should be definitely favoured in order to generate downward (compressive) internal stresses due to seepage at the core-filter exits. Thus we accept that the US-inclined chimney filter is ideal for the embankment. Unfortunately, by a truly unconsidered automatism this has been generally carried down to the foundation, making it a quite unfavourable design decision, shortening the seepage paths of the foundation, and taxing heavily both the unfavourable exit gradients into the filter and the flow conditions of the (unnecessarily lengthened) horizontal drainage blanket.

First and foremost one must make an emphatic statement regarding blankets in general, although it seldom applies to residual soils (in which predominant permeabilities are often vertical). If there is a significant anisotropy (possibly due to significant layering, and in saprolites sometimes favoured by the parent rock structures) resulting in a horizontal permeability much higher than the vertical, the blanket solution would be wrong ab limine (although, as mentioned, the internal blanket would do no harm, could be economical if it uses random compacted earth materials in substitution for rockfill, and may be adjusted to favour compatibility of settlements between the core-filter transitions--rockfill). Flownet solutions (steady-state) for blankets show that there are no benefits from an upstream blanket if the ratios of blanket/foundation permeabilities are high, which is one reason why such a blanket should not be heavily compacted; the other reason, especially important when settlements are expected (e.g. characteristic problem in residual soils) is that a heavily compacted brittle blanket suffers cracking on first reservoir loading. (Fig. 5)

Furthermore, the external blanket very frequently proves to be a wrong solution, subject to serious misbehavior, since the critical loading condition for it is during first reservoir filling (Fig. 6). The published solutions for impervious blankets all assume the steady condition of flownets established both under the blanket and across it (through its vertical thickness). In reality in an unsaturated material it takes much longer for the foundation flownet to reach the steady-state condition which ensures that the blanket need only support a small differential between the reservoir load and the uplift, this being especially serious if the reservoir fills rapidly in comparison with the rate of establishment of the underlying and through-blanket flownets. The advance of the foundation flownet is given by the flow entering under the blanket minus the volume of water taken up in occupying the compression of the air-pores. There can often be some trapped lenses with big air-pores, if some lines of advance of the flownet are faster than others. Serious cases of cracking and punching shear of the blanket have occurred. The trouble has been experienced with greatest ill-effects at about midheight along abutments of unsaturated residual soils. Finally we must reemphasize that these problems are much more serious if settlements under the blanket are big, and especially sudden; and this is typically the critical problem of collapse settlements (item 6) in looser unsaturated residual soils.

The internal blanket solution is characteristically a solution in the right direction as shown in Figs. 6 and 7 , both because it profits of the favourable permeability gradients generated by the compressions, and it employs the pretest or preload principle, since the foundation is more heavily loaded during construction (by the embankment), and when the uplift sets in due to reservoir filling it causes a decrease of effective stresses. (N.B. Unfortunately there can still be some collapse settlements in the more metastable soil structures).

4. Grouted buffers in sound cracked rocks.

Several thoughts have been expressed before (ref. 3,6,7,10) which may be summarized in the following statements:-

1) the water loss pressure test in borings gives a pessimistic (conservative) indication of the need to grout, especially in rocks in which the permeability behaviors of cracks or potential cracks are widely different in the compression quadrant (more frequent in dam foundations) than in the tension quadrant (caused in the test), cf. Fig. 8 ;

2) the water loss pressure test can give good indications of the groutability of a rock, principally if attention is concentrated on determining the probable frequency distribution curves of the coefficients, by using double-packers to pinch-in around a presumed crack, decreasing the test-length;

3) Cement grouting only treats and seals wider cracks, and these are rendered "fully impervious" (cf. Fig. 9). This information can only be confirmed by piezometers carefully installed to tap the specific crack, sufficiently US and DS of the grout line. Obviously, however, the net effect of the grouted buffer in the rock mass is only to reduce somewhat the average flows (average perviousness) it being difficult to detect a significant head differential as would be expected under the wrong hypothesis of a grout "curtain" (thin, essentially impervious, sheet barrier) across the entire rock thickness;(Figs. 10, 11).

4) Single-line grout curtains are adequately effective, but shallow external rows of prior "containment groutholes" can be of interest for reasons of efficiency and economy;

5) Water loss pressure tests are not a direct index for "need of grouting", which is a complex design decision, dependent on the type of rock, type of dam, risk of piping erodibility (or not) along cracks, use of the water saved vs. interest on the incremental cost of the grouting,

and so on. In cracked rocks with a film of in-situ weathered soft clay (clayeyfied joint, as distinguished from clayey infilling), one can reason that even if the films were totally removed by piping, there would still be absolutely no consequence to the dam because of the additional mms. or cms. of settlement. These problems have been postulated under absolutely unreal exaggerated conditions by the prudence of consultants not yet documented with data either from field or laboratory, tests, or from the consequent mental model reasoning.

In view of these points, presently fully documented, the general observation collected from most of our dams on jointed rocks are that we have to introduce more knowledgeable grouting specifications, firstly to improve the efficiency of grouting achievable from each hole (since the principal economy may result from increased spacing between holes and decreased total length of perforation) and to decrease somewhat the rather stringent requirements whereby additional holes have been required to complement the primary holes. Two major representative dams have given total seepage losses of about 190 l/sec and 5 l/sec, which, even if interpreted as wholly due to foundation, are far better than aimed as acceptable. Using data from many dams treated in a fairly analogous but somewhat less stringent manner, a crude index of seepage losses per m of head and per m of crest length has given between 0,01 and 0,05 l/min. m.m. (most frequent) and between 0,0004 and 0,15 l/min.m.m (extremes).

Firstly, in order to improve the radius of action from each hole we should judiciously accept much higher grouting pressures during grout take (with hydrodynamic losses of head, i.e. of uplift capacity, with distance from the hole, during the phases of more flow) and limit carefully the maximum pressure as we reach grout rejection (Freyssinet jack condition). Even for the latter condition it is quite absurd (except in slabby rocks such as sedimentary or metamorphics) to limit the hydrostatic final grout rejection pressure to a hypothetical $\sigma_t \leqslant \gamma z$ (0.25 kg/cm^2 per m of depth, to the packer or to mid-height of the stretch being grouted). The nearest simple theoretical approach to the problem would be by analogy to the extraction (pullout) of anchor plates, although existing solutions only consider geostatic stresses, and in situ residual stresses in rock would offer a benefice difficult to assess. The very rapid decrease of pressure with radius has been recorded in some cases of a hole being grouted when adjacent holes were open (Fig. 12).

Secondly, statistical studies can be undertaken to prove that the principal sealing effect is achieved by the primary-hole grouting (Figs. 13,14,15), and that the requirement of additional holes and grouting based on a grout take (sacks of cement per meter) criterion can be relaxed considerably. The simplest (but somewhat crude) method of statistical comparison of incremental benefices of grouting would assume the statistical universes as represented by so vast a number of data that the sequence of application of the criterion for incremental holes, and its de facto consequence, would be immaterial. For a more prudent check on undesired risks, one would mentally redo the actual sequence of operations and, in applying a less stringent criterion, would mentalize what zones would remain untreated. The immediate urge would be to apply deterministic truncations, but if the rock is considered a statistical universe, a further step would consider such truncations as regulated by probabilities.

Considerable economies can be foreseen but considerable additional work and judgment must be applied in selecting for each site the appropriate initial spacing of primary holes and appropriate grouting pressure for satisfactory initial grout travel.

5. Specialized grouting in saprolites and laterites.

The experience with specialized grouting of saprolites and laterites arose recently with the desire to treat erratic holes (canaliculi, of mm to about 10cm diameters) that abound in some laterite horizons to great depths (20-25m) in a manner as yet rather indeterminate, unpredictable, and unexplained. One hypothesis is that their origin is associated with ancient termite colonies (Refs. 1,16). Whatever the hypothesis, it is obvious that the probabilities of some holes (canaliculi) being found by other holes (borings or groutholes) are most remote: the principle of successful grouting of cracked rocks is that it is reasonable to anticipate groutholes crossing planar cracks (joints etc.) and thereupon the grout travel (liquid) ensures taking up the open planes crossed. In a reasoning of direct antithesis it was planned to create planes of hydraulic fracturing by the grouting: thus, not only would the horizon be generally benefited in imperviousness by the grouted planes, but also the canaliculi that would be perchance crossed by the grouting planes, would also have adequate opportunity of serving for some grout travel and consequent sealing. The improvement of the unsaturated soil mass (any saprolite irrespective of lateritic canaliculi) crossed by grouted planes of hydraulic fracturing is twofold: partly there is a compression (consolidation grouting) of the soil volume by thickening hydraulic fracture widths; and partly, each grouted plane would itself act as a relatively impervious sheet, all directions of planes within the treated zone (hopefully somewhat crisscrossing) being favourable except those that would probabilistically lie vertical in directly upstream-downstream directions.

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

It is beyond the intentions of this paper to delve into the developmental details that gradually achieved a treatment apparently meeting the foundation design requirements, as proven by careful inspection of the faces of several trenches, both during grouting and grout emergence, and after adequate set of the several grouts. Differently coloured clay--cement mixes were used, both to differentiate grouts forced out of different manchetes, and to differentiate grouts forced out of the same manchete in distinct stages. Some of the interesting data collected from the extensively observed trials are somewhat documented in the tables, graphs and photographs (Figs. 16, 17). In short, selective grouting by hydraulic cracking emanating from manchetes in the tube-a-manchete technique has been successfully achieved. In some horizons (principally the upper residual soil) the cracked planes were most frequently vertical, but in the saprolites, frequent cracked planes were inclined and parallel, suggesting σ_3 planes associated with relict joints. One curious observation was that quite repeatedly successive phases of grouting developed planes very near to previously grouted ones, even when the grouted thicknesses reached as much as 1-2 cms: it would thus seem that despite significant compressions, the preferential planes of minimum σ_3 may not change. In the case of a continuous grouting with successive coloured grouts, it was found that the tongue of new grout wedges in the center of the previous grout, pushing it outwards. Many canaliculi were encountered well grouted, as desired: in some cases where apparently the grouting did not extend long enough to fill canaliculi completely, it was found that the partial grouting formed a tubular infilling, with central hole. The criteria for grout volumes and pressures intended to limit travel of cracking and grouting, still need judicious adjustments: it has been found for the present that at the pressures and rates of pumping used, the grout pushes forward essentially with no loss of head, and therefore stoppage has to be provoked either by an arbitrary limitation of grouted volume, or by significant reduction of pressure. Clay-cement mixes have been achieved that are remarkably compatible with the surrounding soil, as judged by meticulous tactile inspection cutting across the inspection pit faces with a pocket-knife.

The important point is that one does not need to visualize the impervious treatment of such saprolites and laterites as including, as only alternates, either the excavation and recompaction of very deep cutoff trenches, or the execution of deep clay-cement slurry-trench cutoff walls extended by cement grouting of the underlying rock, or the use of the yet relatively expensive chemical grouting possibilities (Ref. 10).

6. Collapse settlements in residual soils.

The metastable nature of porous unsaturated residual soils has been recognized since the days of the Santa Branca, 55m (1954), and Tres Marias, 70m (1958), dams. Figs.18,19 summarizes the data from the latter dam, where it was reasoned that collapse settlements would be small and unimportant, especially where the compressions due to embankment overburden would have been greater, absorbing much of the collapse settlement potentiality. The Tres Marias left abutment settled as much as 1.25 to 1.35m during construction, almost instantaneous, with less than 10% incremental settlement (possibly in part due to stress adjustments) with time: there was no noticeable collapse settlement upon reservoir filling (at the higher sections measured), which was a little surprising and fortunate. Apparently the very smooth variation of conditions along the abutment, alongside with the very conservative embankment section, joined in providing a satisfactory behavior, despite the overly simplified reasonings and decisions.

Figs.20,21 presents data on the Paraitinga dam (110m, 1977) which suffered sudden collapse settlements over a limited area, overlying a medium dense saprolite of the left abutment: there was some surface cracking, first noticed in the concrete-paved surface drainage trenches; but it was concentrated in the downstream slope, and the generous upstream homogeneous impervious section did not suffer at all. One may reffect on questions of planned instrumentation locations and erratic natural occurrences vs. tendencies to redistribution of stresses and strains. The fact is that collapse tends to be "instantaneous" upon wetting or soaking (loading previously applied maintained constant), although saturation may be very slow or even never attained.

The problem of collapse settlements can well be the most serious problem of core-dams founded on residual soils and saprolites. This is so for two reasons: firstly, the fact that these settlements are not provoked either by static pressures alone or by soaking (submergence) alone, but by critical combinations of the two; secondly, that these settlements are sudden, therefore most unfavourable for causing cracking of the finished dam due to differential settlements. Such cracking, if transverse, across cores of but modest width could generate problems.

In the case of the Guri dam, the interference of metastable collapse problems was discovered after the Bid Documents had been out

(June 1976) and the extreme solution was taken of a major change of axis, with significant increase of compacted fill volumes, to avoid the undesirable foundation areas. Several papers published (Refs. 17,18,19,20) all repeatedly concentrated on the questions of soil tests on the significantly collapsive granitic saprolites despite their high SPT blowcounts. Of course, it can be readily seen that oedometer tests (presumed confined to zero lateral strain) are pessimistic (by not achieving this condition, nor controlling lateral stress), and plate load tests are noticeably irrelevant because of the major interference of bearing capacity loading conditions: only judicious and careful triaxial tests can approach relevance to the desired parameters.

However, it is once again emphasized that to achieve an appropriate engineering solution, attention must be directed at means to avoid the problem and/or to apply "pretest conditions", so that the dam will not suffer sudden changes at the inappropriate (first filling) time (Design Principle DP-4, Rankine Lecture 1977). In wide very conservative "homogeneous" sections the problem reduces in significance considerably, because it is not difficult to foresee that across any transverse section widths of uncracked material of more than 0.2H should have good probabilities of remaining as a sinuous, non-geometric, impervious core. At any rate, even in narrow core dams with US and DS rockfill sections one can well adopt engineering solutions so as to guarantee that the collapse settlements (small increment over the total settlement) occur during construction. Such should be the desirable engineering trend, technically and economically: it is very much more economical than adopting changes of axes or of embankment sections.

The adjustment of soil testing to represent the engineering behavior is a second step, of soil engineering scientific endeavour, which does not preclude or supplant the engineering design step ab limine.

Collapse settlements are probably the principal crucial problem of embankment dams on residual soils and saprolites. The engineering solution to this problem has been developed but awaits prototype confirmation.

REFERENCES

BARROS Machado, A. "A exploração biológica da Lunda", Academia das Ciências de Lisboa, Lisbon 1968.

BERNEX. "Étude des variations de la perméabilité des roches sous contraintes. Thèse de doctorat, 1966.

De MELLO, V.F.B. "Some quantitative investigations on curtain grouting in rock foundations of earth dams". First Panam. Conf. SMFE, México, Sep. 1959, v. II, p.699.

De MELLO, V.F.B. Discussion: "Earth and rock dams, slopes, and open excavations". Sixth ICSMFE, Montreal, Sep. 1965, v. III, p.577.

De MELLO, V.F.B. "Some lessons from unsuspected, real and fictitious problems in earth dam engineering in Brazil". Sixth Regional Conf. SMFE, Durban, Sep. 1975, v. II, p.285.

De MELLO, V.F.B. "Reflections on design decisions of practical significance to embankment dams". Rankine Lecture, Géotechnique 27(3), 1977, p.279.

De MELLO, V.F.B. "Practice, precedents, principles, problems, and prudence in embankment dam engineering" - Symposium on Problems and Practice of Dam Engineering, Bangkok, Dec. 1980.

De MELLO, V.F.B. "Comparative behaviors of simular compacted earthrock dams in basalt geology in Brazil". Symposium on Problems and Practice of Dam Eng. Bangkok, Dec. 1980.

De MELLO, V.F.B. "Design trends on large rockfill dams and purposeful monitoring needs". Int. Symposium on Field Measurements in Geomechanics, Zurich, Sep. 1983.

De MELLO, V.F.B. "Concrete gravity dam foundation: an open of geomechanical interaction, structure - foundation and theory-practice". Fourth Australia - New Zealand Conf. on Geomechanics, Perth, May 1984, v. I, p.177.

De MELLO, V.F.B. Experience and data from main Brazilian dams - Private technical files.

QUEIROZ, L.A. "Compressible foundation at Três Marias earth dam". First Panam. Conf. SMFE, México, Sep. 1959, v. II, p.763.

MARSAL, R.J. & RESENDIZ, D.N. "Effectiveness of cutoffs in earth foundations and abutments of dams". Fourth Panam. Conf. SMFE, Puerto Rico, June 1971, v. I, p.237.

MARSAL, R.J. & RESENDIZ, D.N. Presas de Tierra y Enrocamiento. México, 1975, 546p.

MAYER, A. "La mécanique des roches". Sixth ICSMFE, Montreal, Sep. 1965, v. III, p.104.

PENMAN, A.D.M., et al "Sandstone rockfill in two dams". Fourteenth ICOLD, Rio de Janeiro, May 1982, v. IV, p.279.

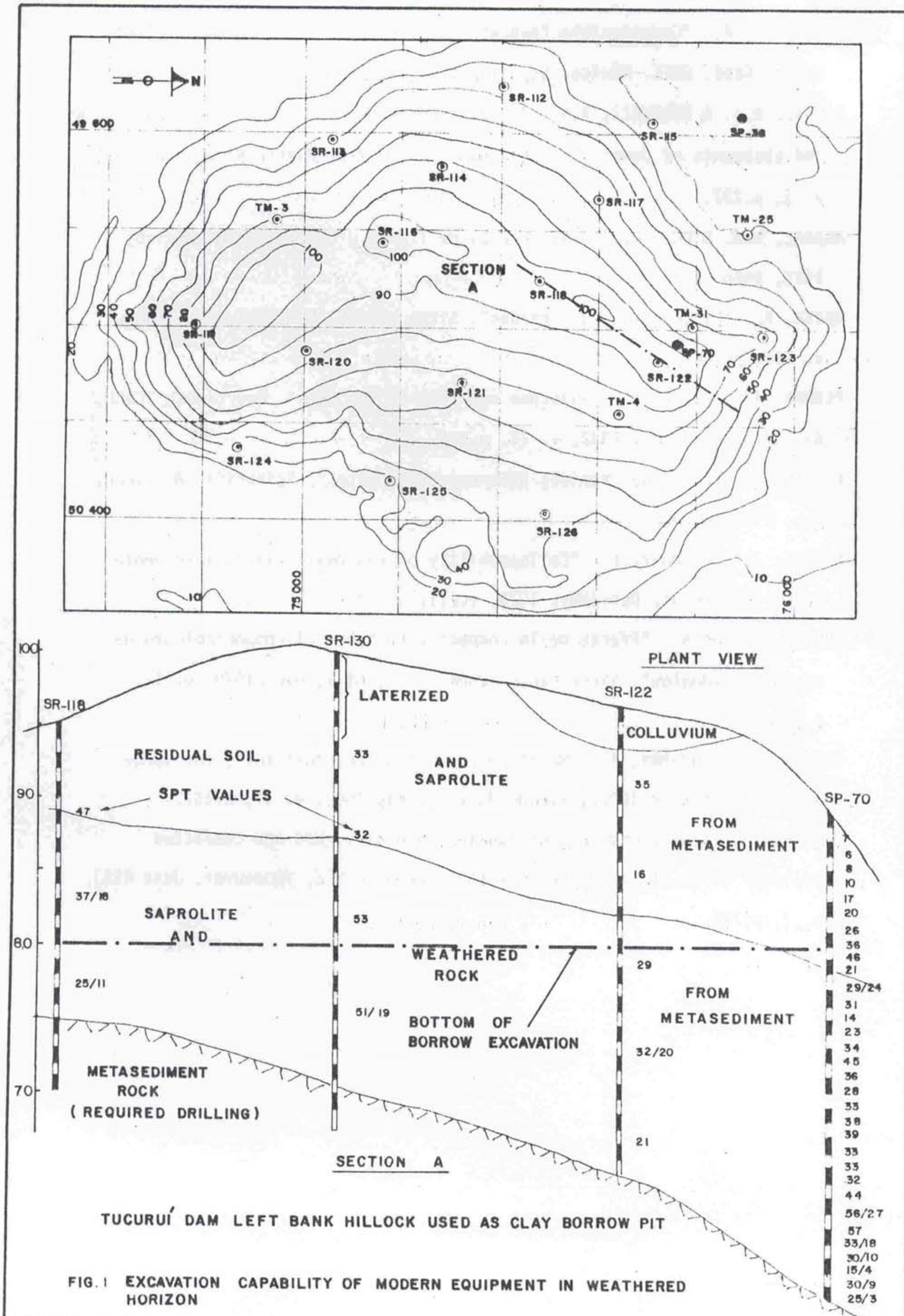
PRESTWICH, G.D. "The chemical defenses of termites". Scientific American, Aug. 1983, p.68.

PRUSZA, Z. & CHOUDRY, T. "Collapsibility of residual soils". Thirteenth ICOLD, New Delhi, Oct./Nov. 1979, v. II, p.117.

PRUSZA, Z., et al "Efecto de la compactacion sobre el comportamiento de suelos residuales". Sixth Panam. Conf. SMFE, Lima, Dic. 1979, v. II, p.377.

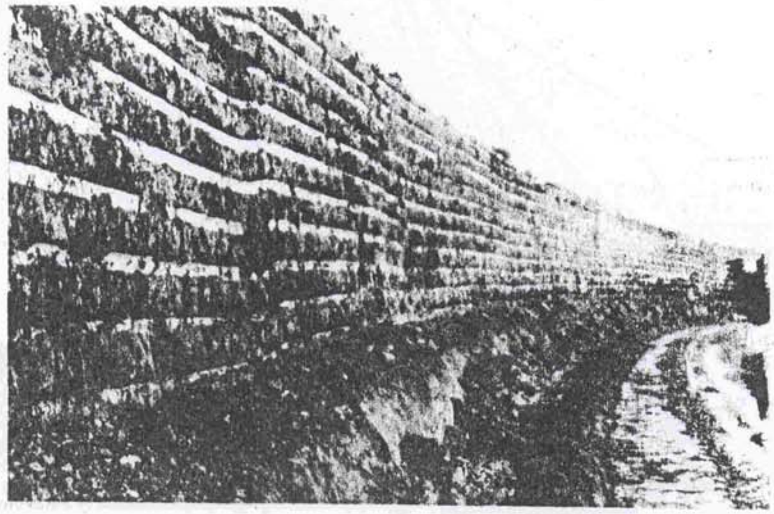
PRUSZA, Z. & CHOUDRY, T. "Moisture control of residual soils for large dams". Fourteenth ICOLD, Rio de Janeiro, May 1982, v. IV, p.167.

PRUSZA, Z., et al "The compressibility of undisturbed and compacted residual soils at Guri". Seventh Panam. Conf. SMFE, Vancouver, June 1983, v. I, p.185.

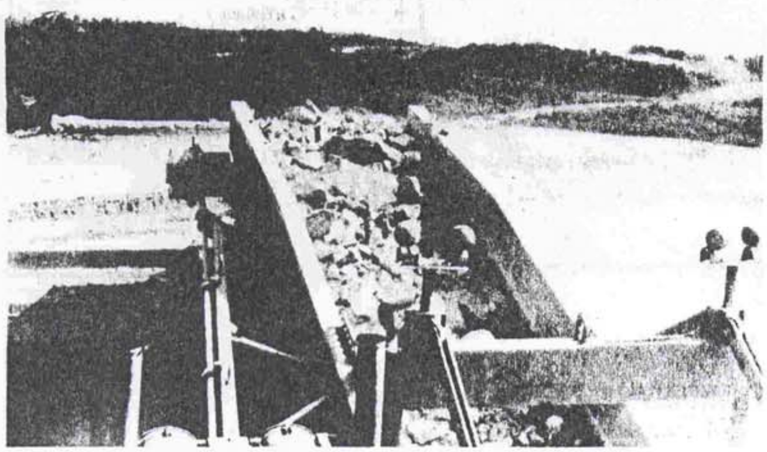


TUCURUÍ DAM LEFT BANK HILLOCK USED AS CLAY BORROW PIT

FIG. 1 EXCAVATION CAPABILITY OF MODERN EQUIPMENT IN WEATHERED HORIZON



2a - EXCAVATED FACE BY LOADER: TRANSITION FROM DENSE SAPROLITE (SPT $\geqslant$ 30) TO MEDIUM DENSE WEATHERED ROCK.



2 b - CHUNKS OF SAPROLITE AND WEATHERED ROCK AS DEPOSITED IN WAGONS FOR HAUL TO DAM.

FIG. 2 USE OF DENSE SAPROLITE AND WEATHERED ROCK FOR COMPACTED IMPERVIOUS CORE.

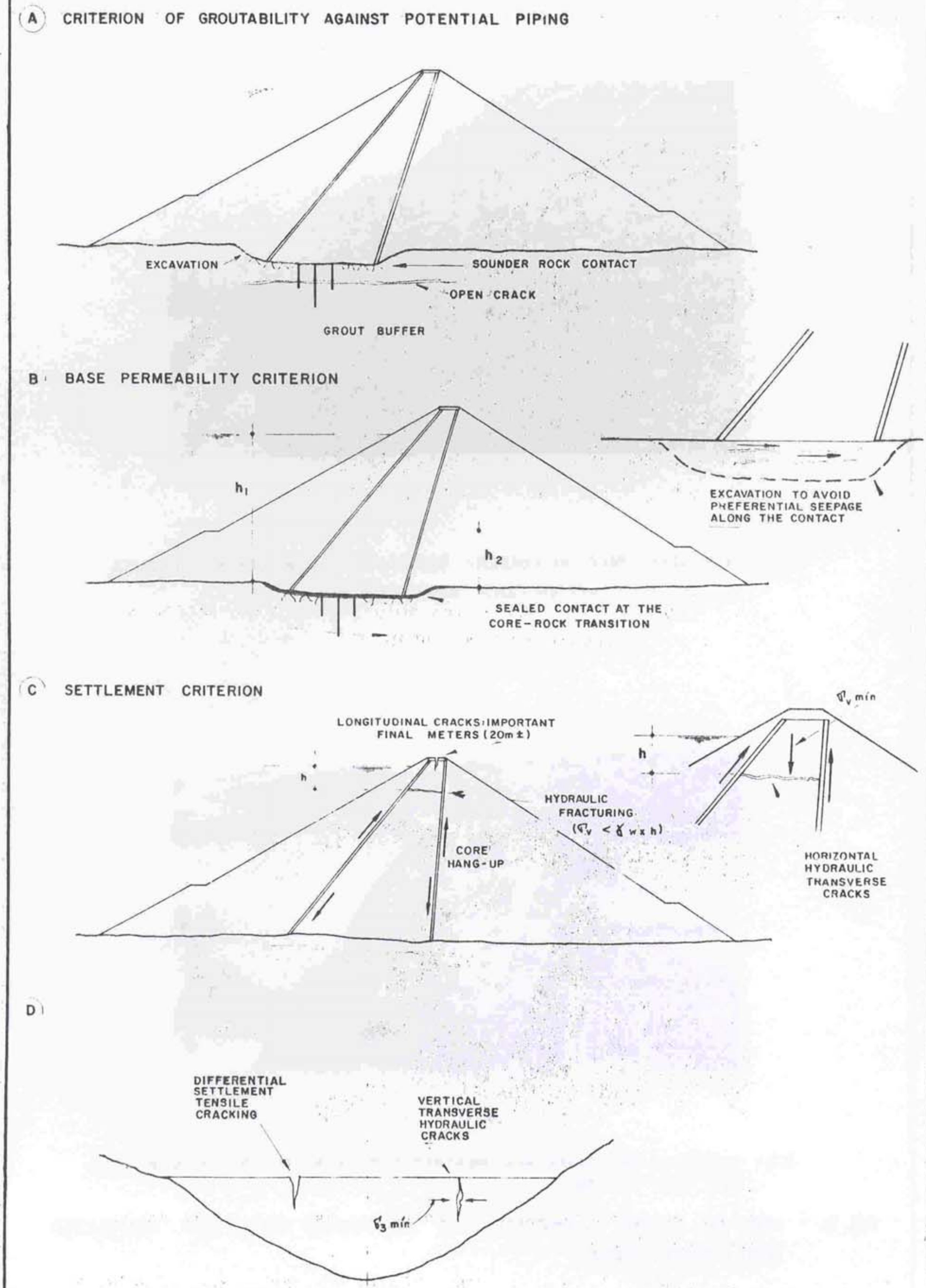


FIG. 3 SCHEMATIC SUMMARY OF PROBLEMS POSSIBLY GENERATED BECAUSE OF FOUNDATIONS UNDER CONSIDERATION

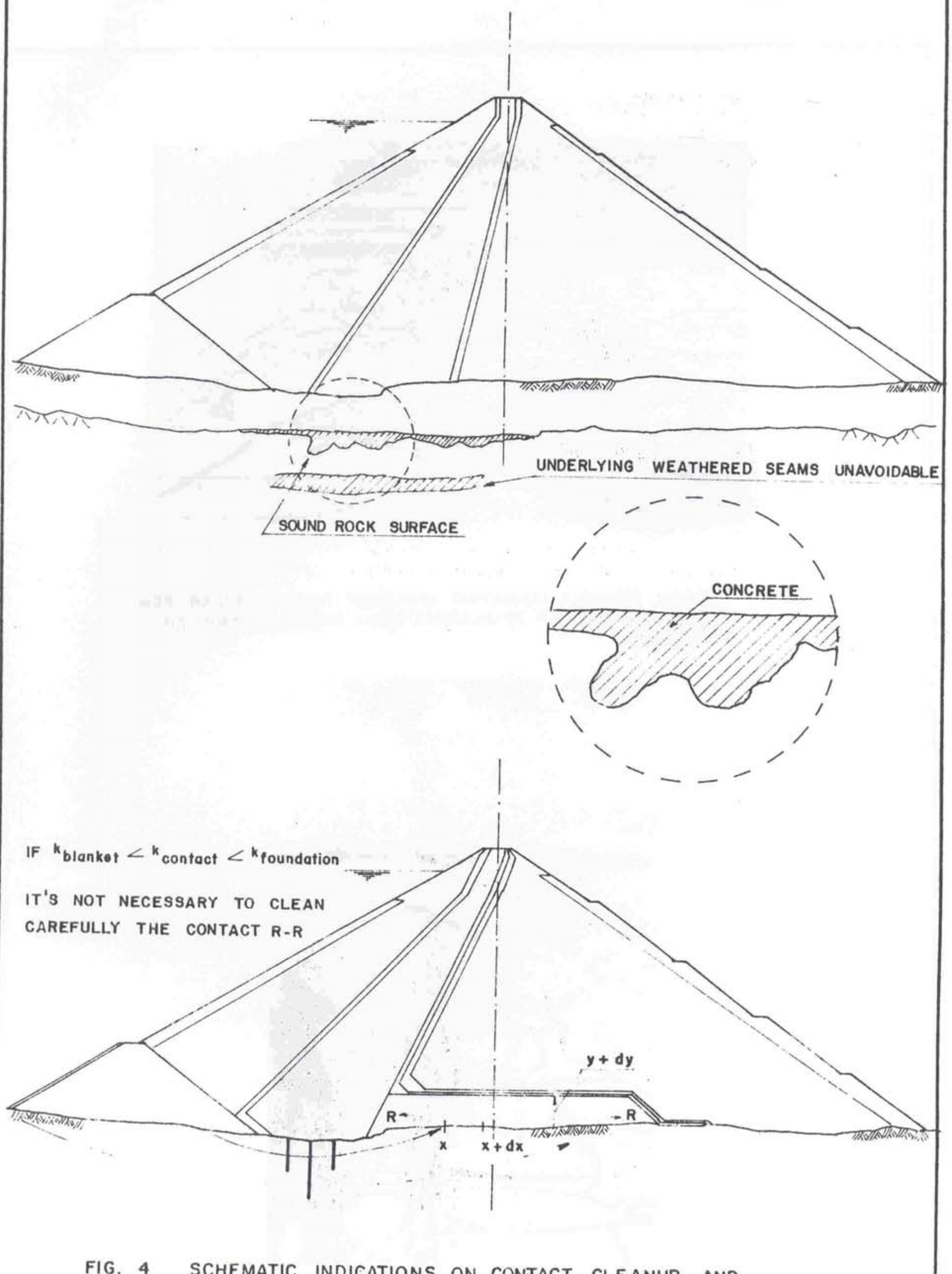
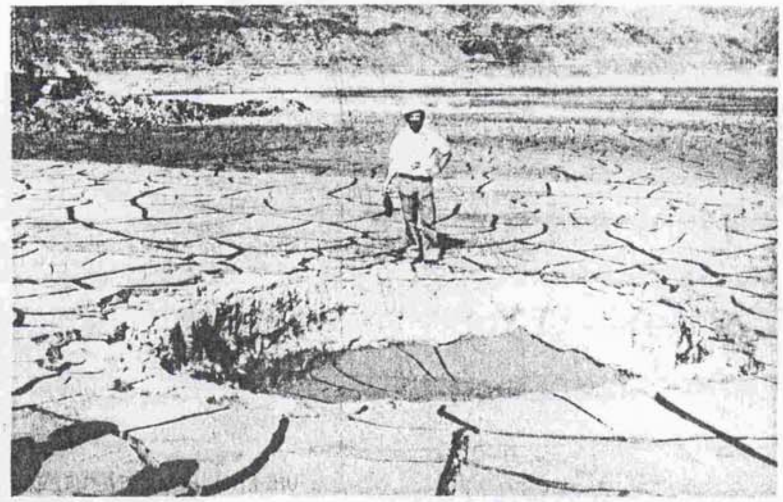
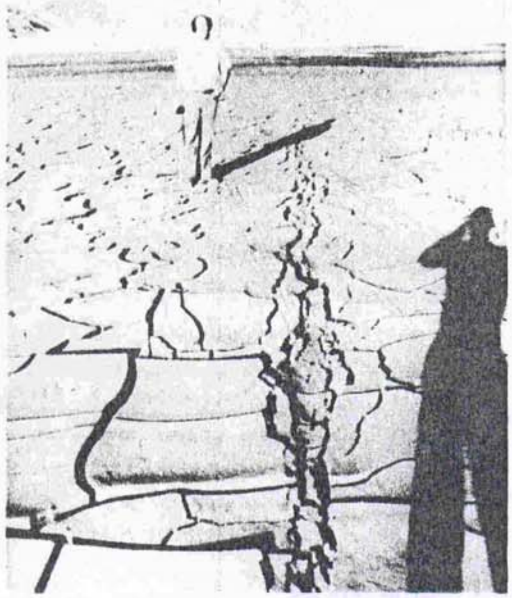


FIG. 4 SCHEMATIC INDICATIONS ON CONTACT CLEANUP AND TREATMENT NEEDS.



5 a- TYPICAL SINKHOLE INDICATING APPARENT PUNCHED SHEAR .FEW HUNDREDS OBSERVED IN BLANKET WHEN RESERVOIR EMPTIED BY FAILURE



5 b- TYPICAL CRACK OBSERVED IN BLANKET

FIG. 5 PROBLEMS FROM RAPID RESERVOIR LOADING ON UPSTREAM BLAKET.

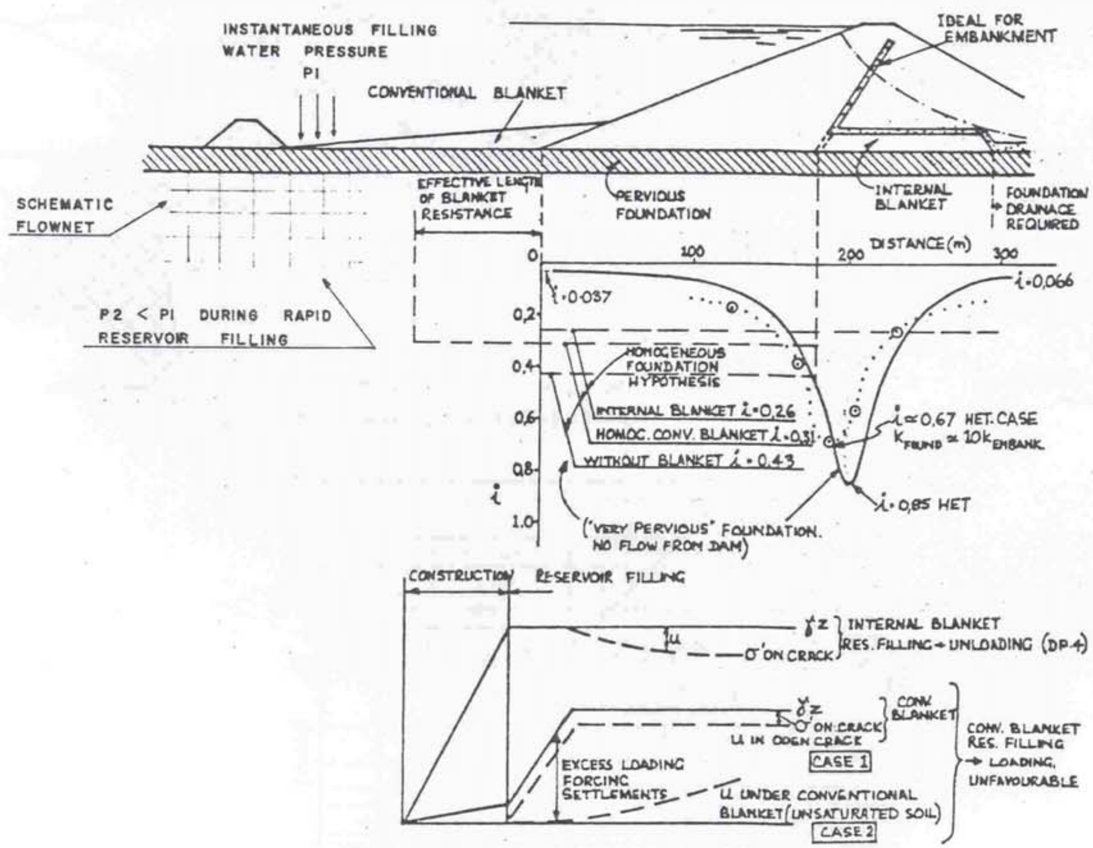


FIG. 6 ADVANTAGES OF INTERNAL IMPERVIOUS BLANKET REGARDING SEEPAGE GRADIENTS IN FOUNDATION AND LOADING SEQUENCES

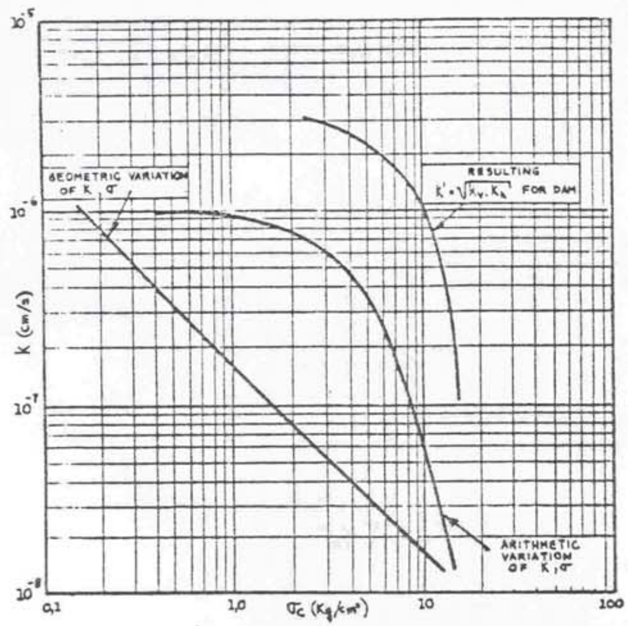


FIG. 7a CHANGE OF PERMEABILITY WITH CONSOLIDATION. HETEROGENEOUS (HET) DAM.

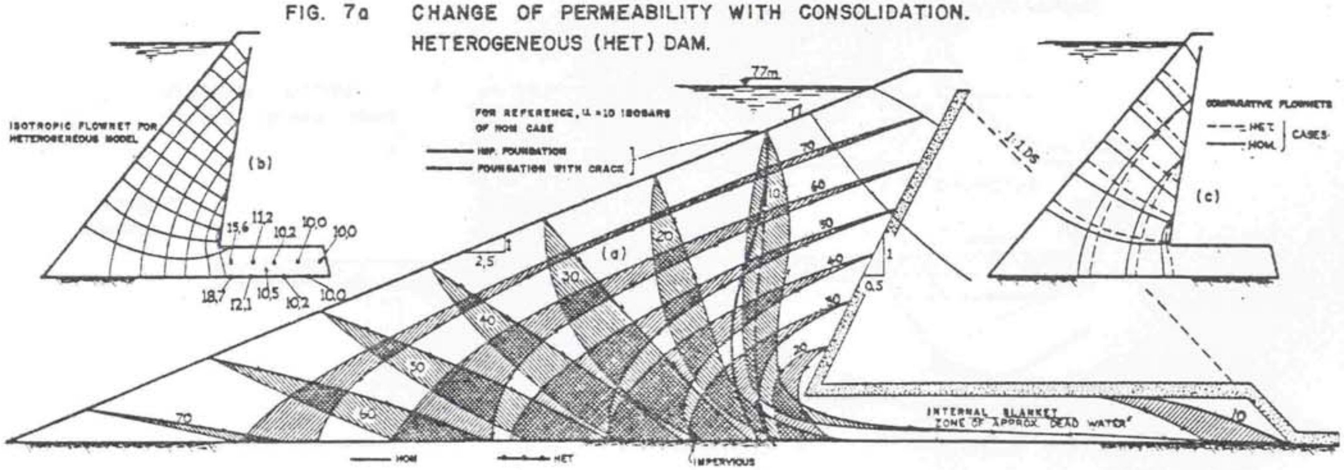


FIG. 7b DESIGN RECOMMENDATION OF INTERNAL BLANKET

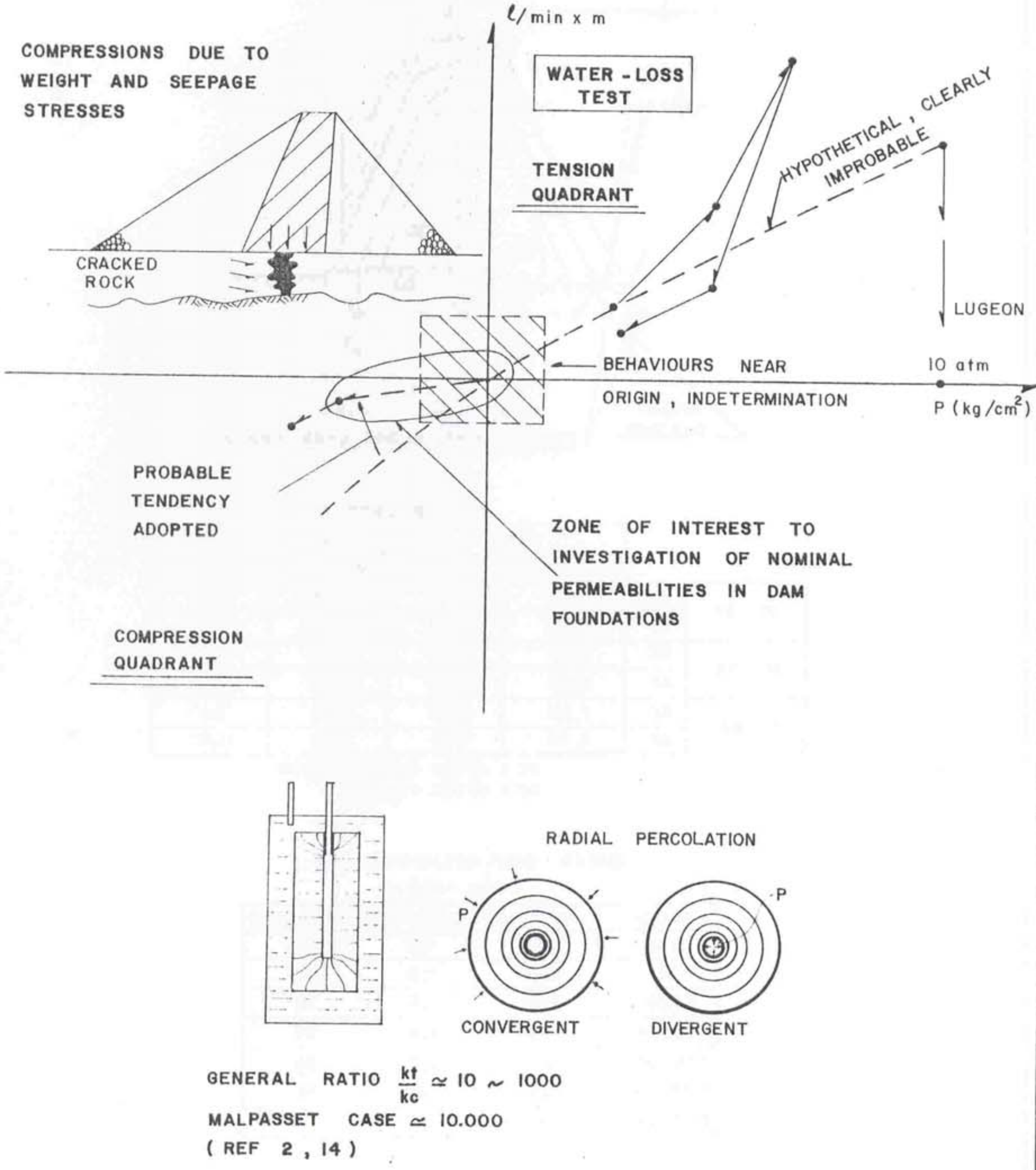
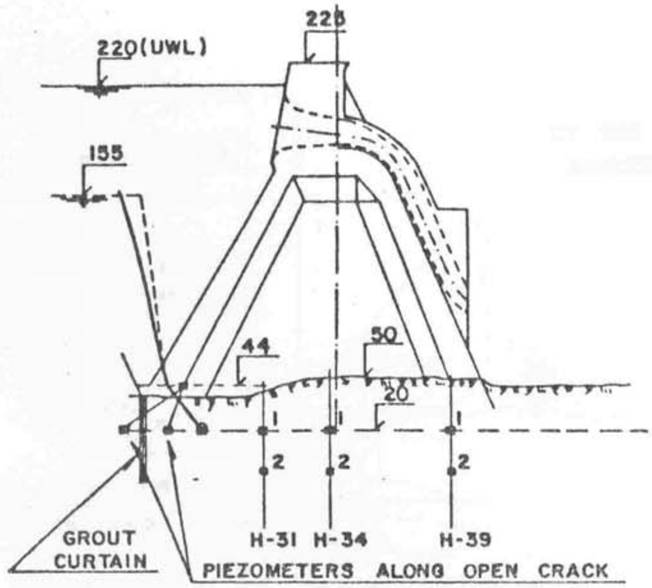


FIG. 8 INTERPRETATION OF WATER - LOSS TESTS - TENSION AND COMPRESSION QUADRANTS



SETTLEMENTS (mm)

		ANCHOR 1		ANCHOR 2	
		OBSERVED	PREDICTED	OBSERVED	PREDICTED
H : 31	AC	0,63	11,7	0,45	8,6
	AF	0,70	10,0	0,50	7,4
H : 34	AC	3,15	11,5	2,35	9,2
	AF	3,65	10,9	2,80	8,0
H : 39	AC	1,05	10,0	0,75	7,4
	AF	1,70	15,0	1,35	11,0

AC = AFTER CONSTRUCTION
AF = AFTER FILLING

SHEAR DISPLACEMENTS (mm)
(UWL = 220 m)

BLOCK	HIGH (m)	OBSERVED	PREDICTED
E 6	85	1,0	5
F 1/2	102	1,0	20
F 5/6	130	1,6	17
F 13/14	185	1,4	36
F 19/20	185	2,3	36
F 27/28	135	1,6	16
F 35/36	110	1,3	11

FIG. 9 ITAIPU DAM, FOUNDATION OBSERVATIONS, OBSERVED vs. PREDICTED.
NOTE: HEAD DROP FROM US TO DS OF GROUTING

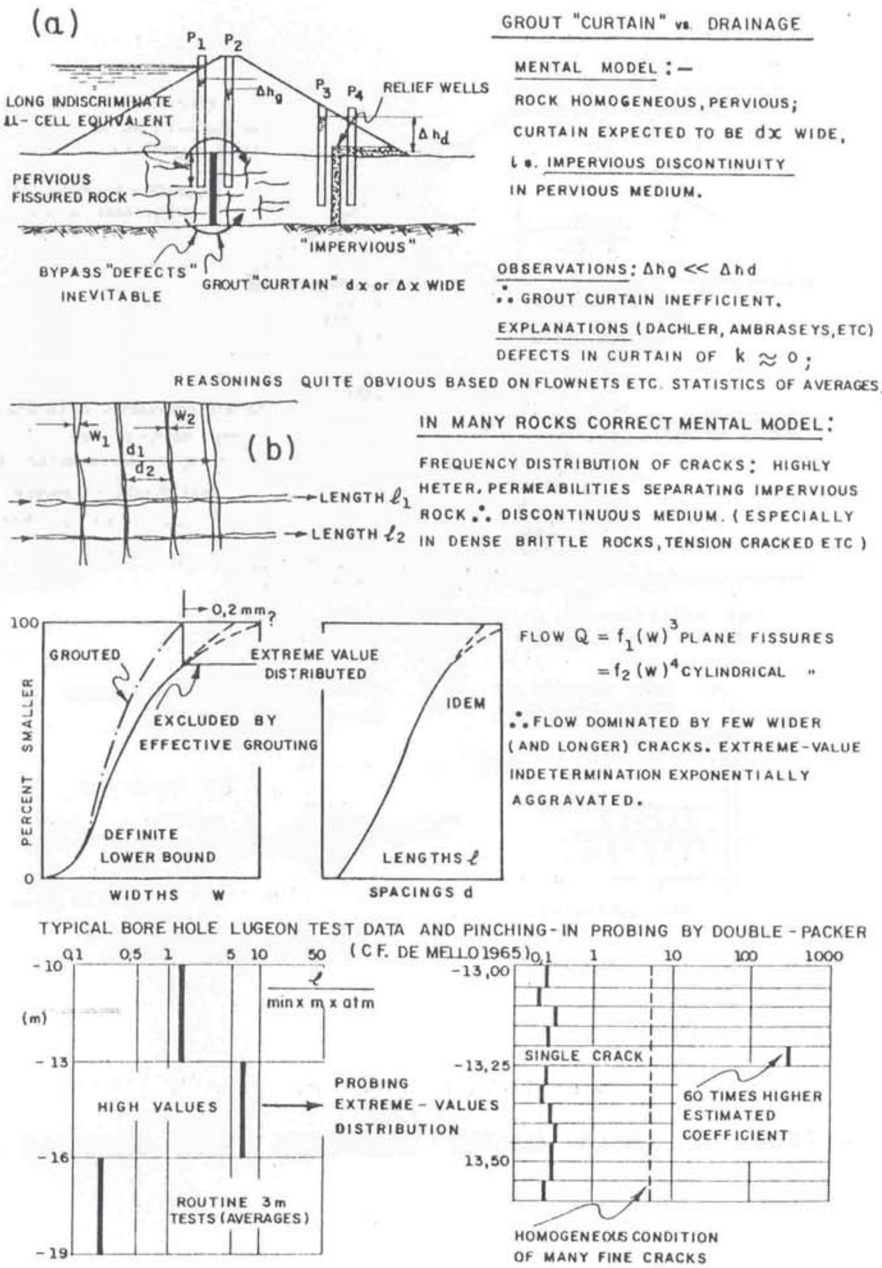


FIG. 10 GROUTING OF FISSURED ROCK VIEWED AS HOMOGENIZING TREATMENT ATTEMPTING TO EXCLUDE EXTREME-VALUE CONDITIONS

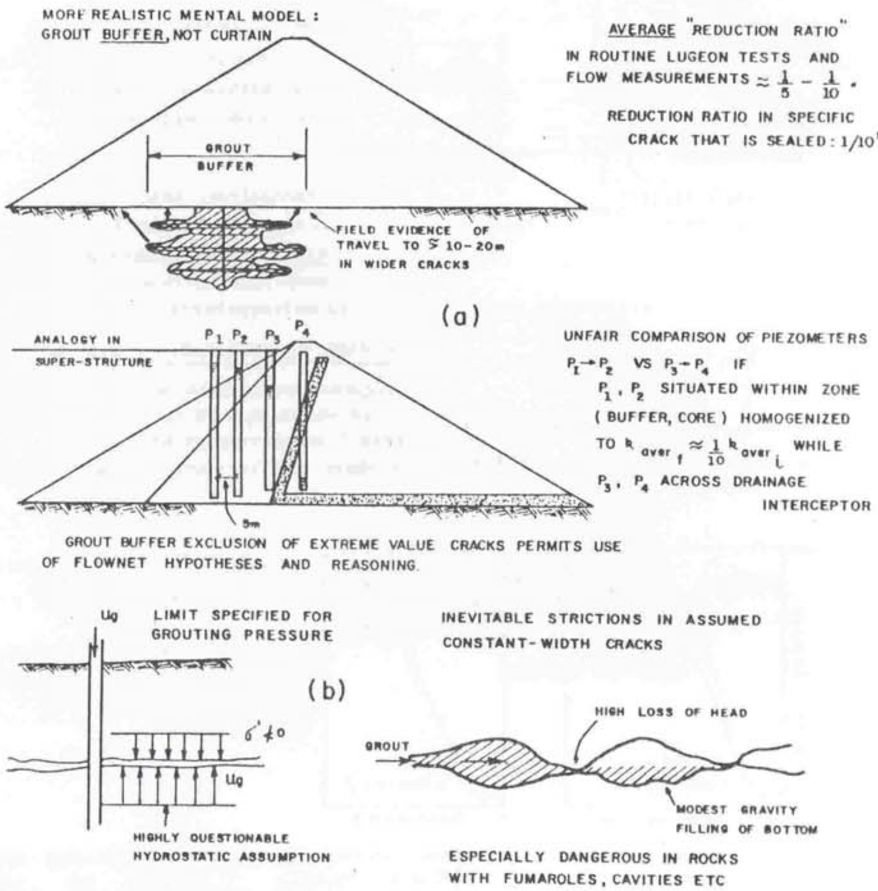
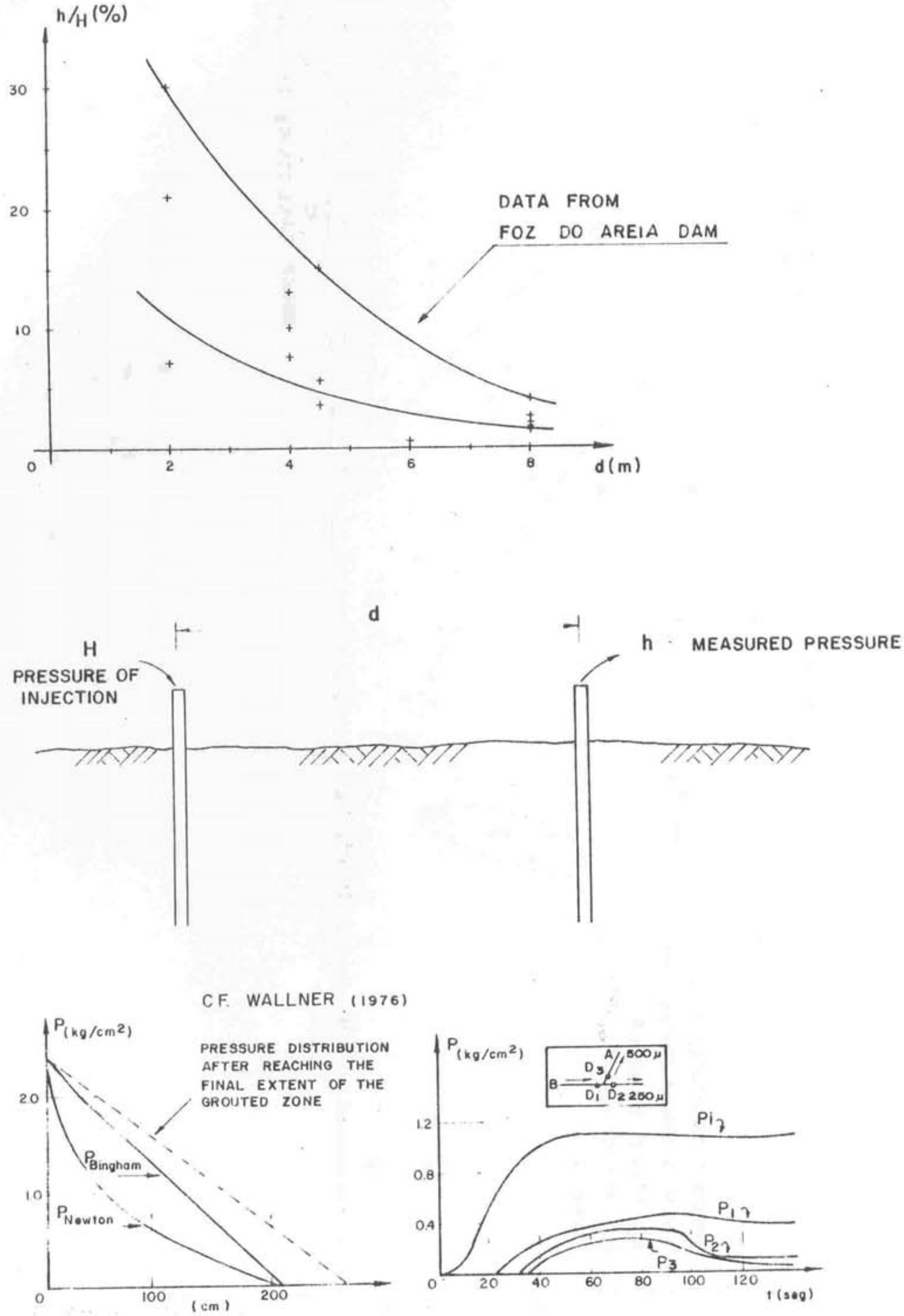


FIG. 11 EFFECT OF GROUT BUFFER COMPARED WITH DRAINAGE INTERCEPTOR



PRESSURES DURING GROUT PROPAGATION (REF 6)

FIG. 12 PRESSURE DROP WITH DISTANCE FROM GROUTING HOLE

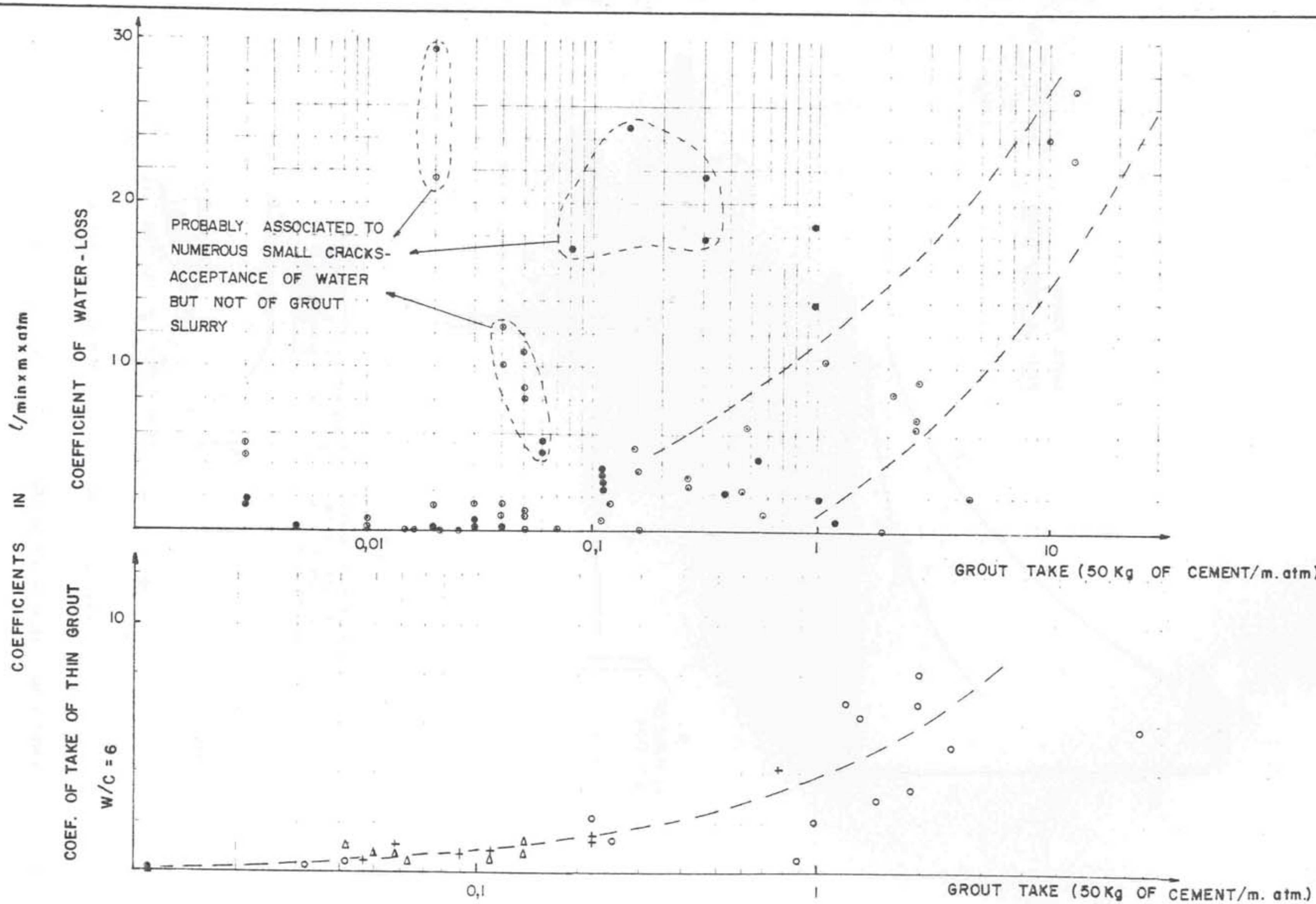


FIG. 13 EMBORCAÇÃO DAM. ATTEMPT AT CORRELATIONS OF TEST COEFFICIENTS VS. GROUT TAKES

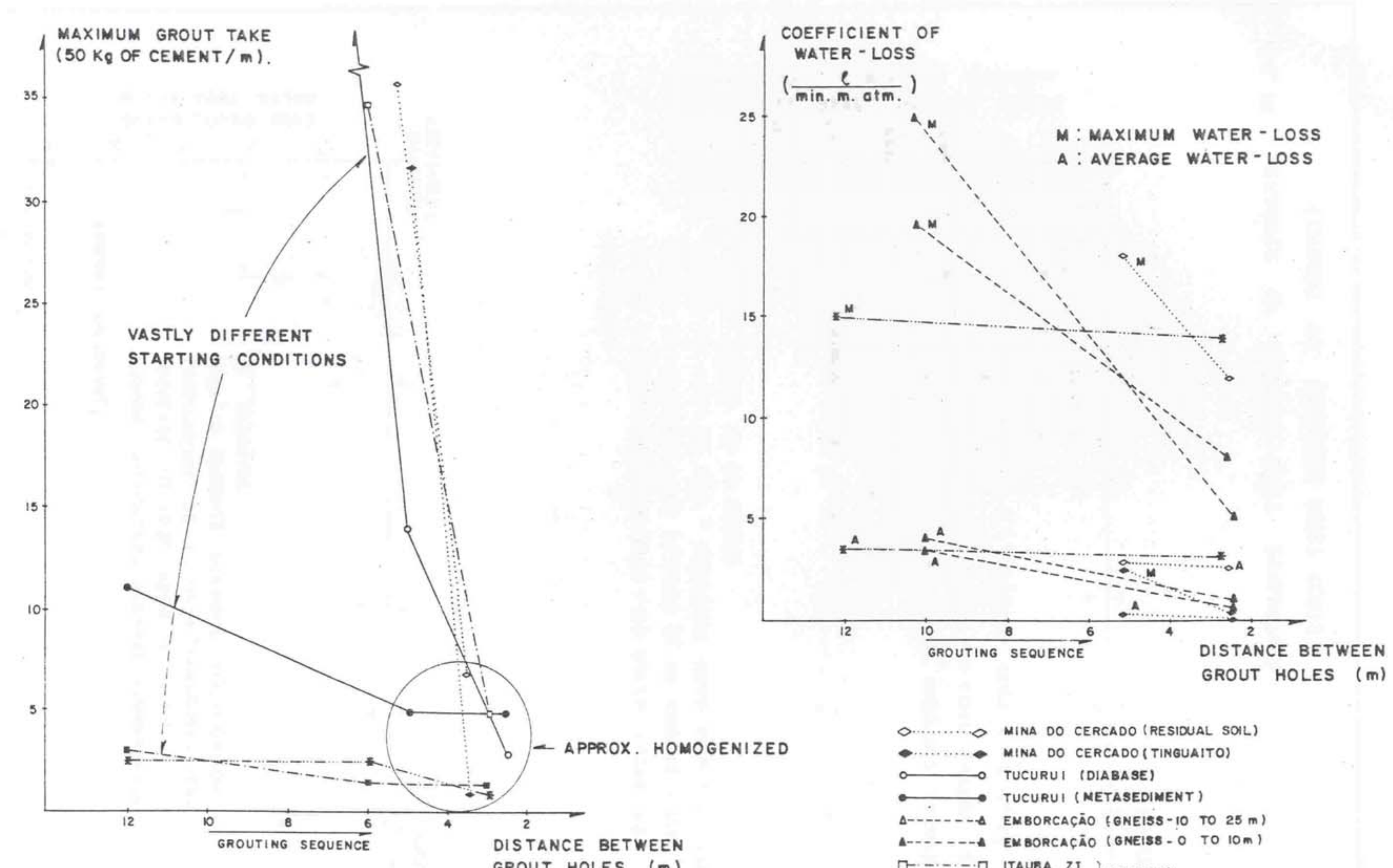


FIG. 14 SEQUENTIAL GROUTING INDICATES DOMINANT EFFICIENCY OF FIRST HOLES GROUTED; ONLY REQUIRE ADJUSTMENT OF STARTING SPACING

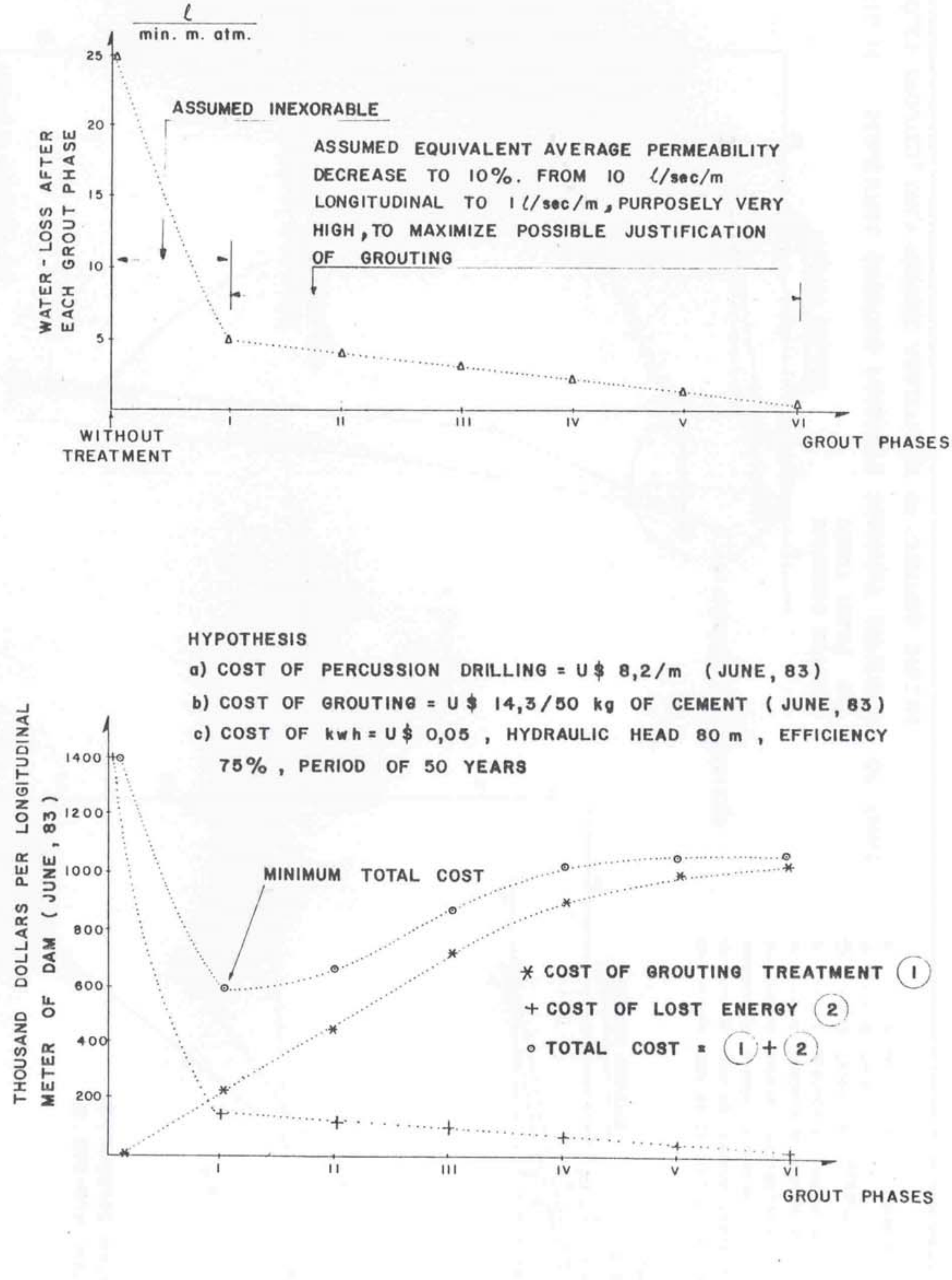
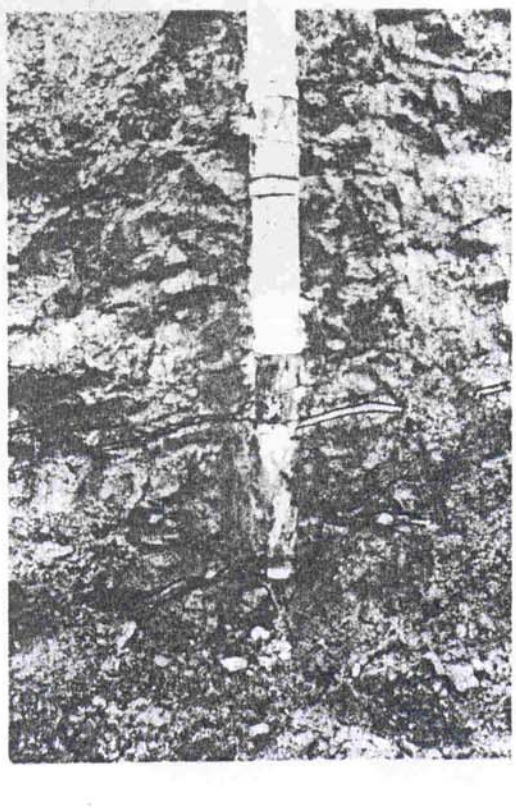
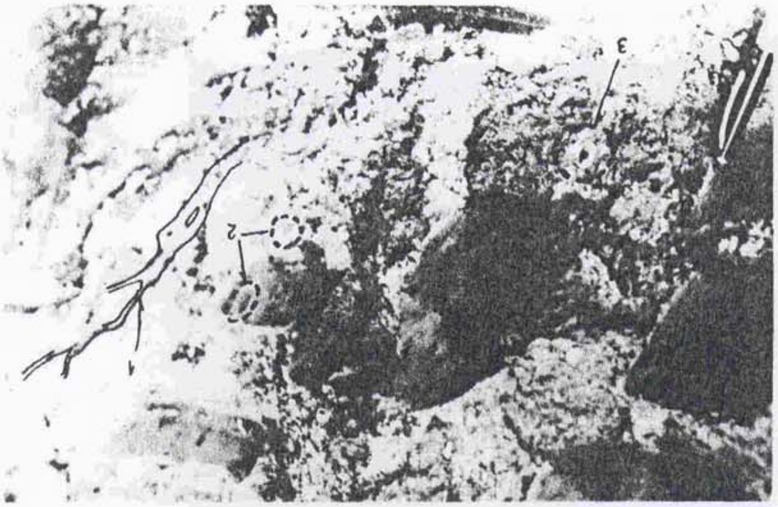


FIG. 15 EXAMPLE OF BENEFIT - COST EVALUATION
(ENERGY VS. GROUTING FIRST COSTS)

16 a - FIELD TEST LAYOUT TO CHECK GROUT TRAVEL AND PRESSURES RECORDED AT DIFFERENT RADII.



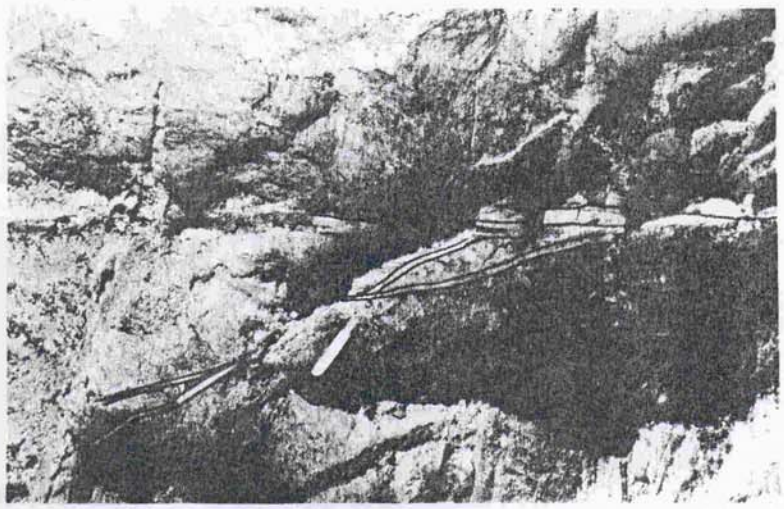
16 b - TYPICAL TUBE-A-MANCHETE GROUTHOLE IN SAPROLITE. PVC, AND RUBBER SLEEVES.

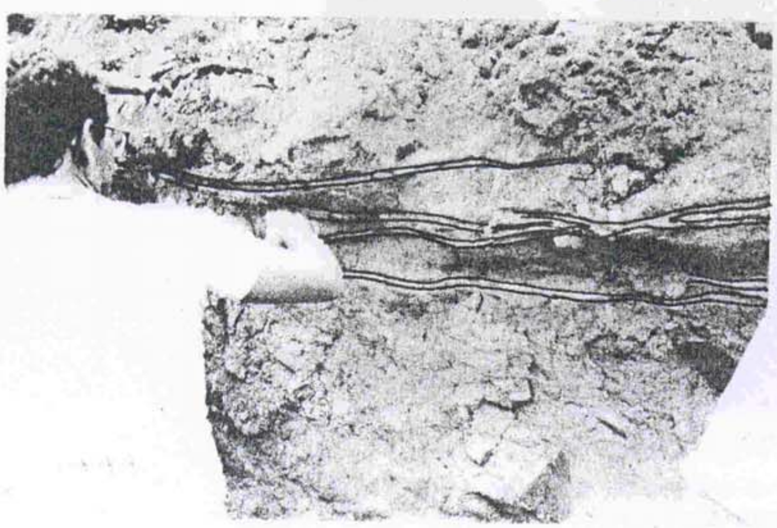


a -

1 GROUTED HYDRAULIC FRACTURES
2 FILLED CANALICULUM
3 CANALICULUM WITH TUBULAR GROUT

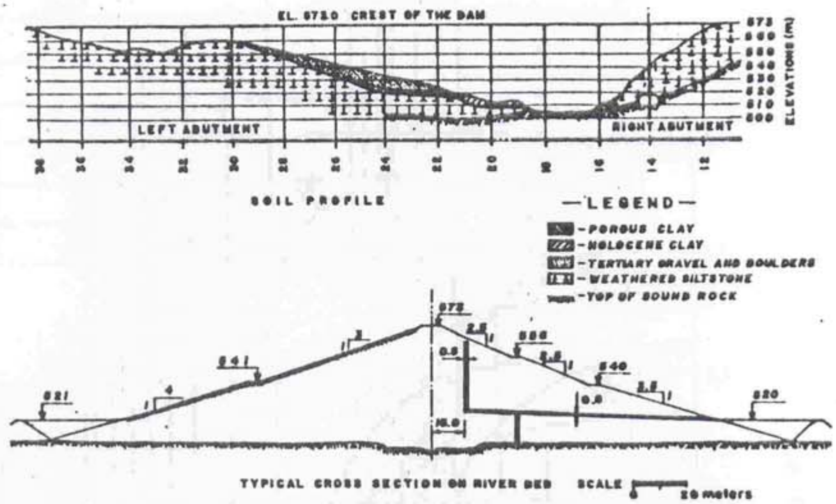
b - GROUTED HYDRAULIC FRACTURES, DIFFERENT INCLINATIONS



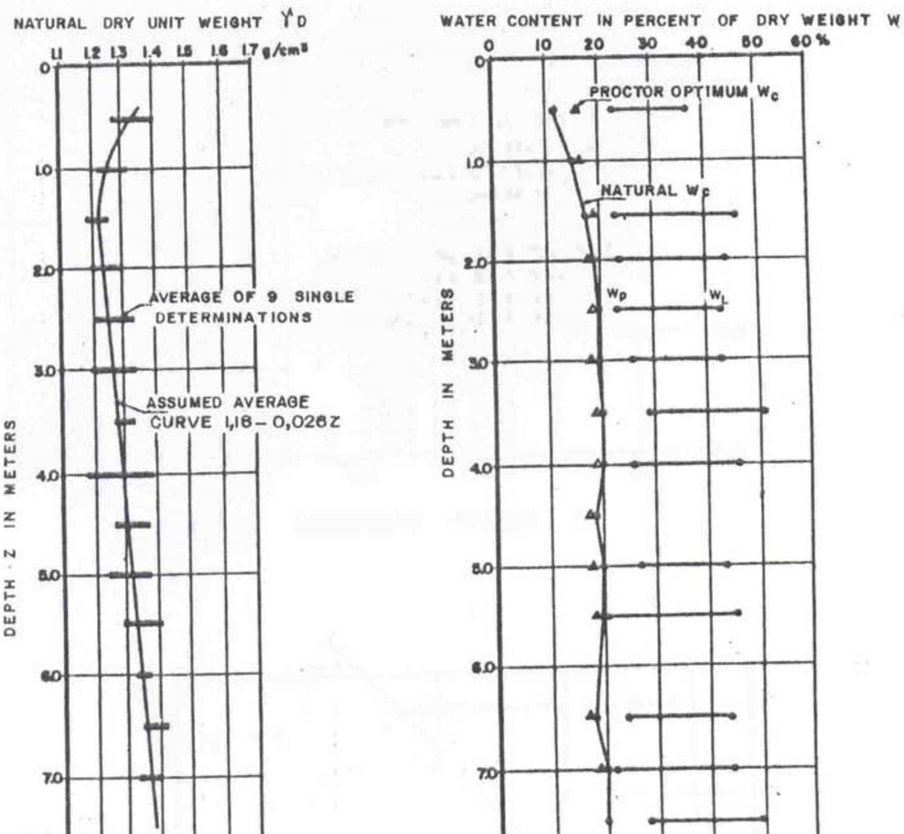


c - GROUTED HYDRAULIC FRACTURES, HORIZONTAL

FIG. 17 EVIDENCES OF PREFERENTIAL GROUTING IN LATERITE AND SAPROLITE; INSPECTED TRENCH



a) TRES MARIAS DAM, 1959, 70 m. GENERAL FEATURES. POROUS RED CLAY UP TO 12 m THICK, LEFT ABUTMENT



b) COLLAPSIVE POROUS RED CLAY (RESIDUAL OF SEDIMENTS). PROFILE OF INDEX PROPERTIES

FIG. 18 CASE-HISTORY OF DAM ON COLLAPSIVE SOIL

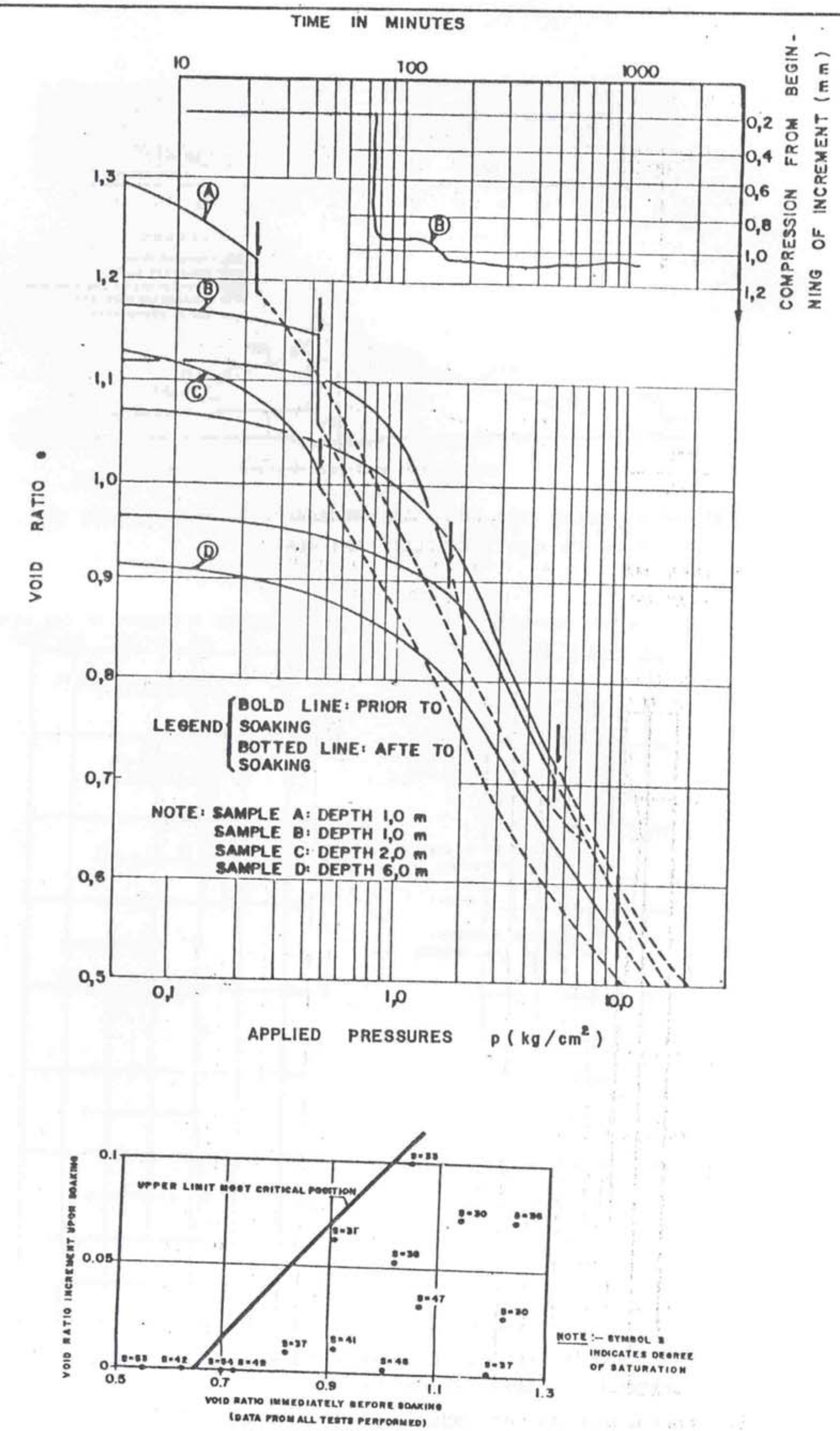


FIG. 19 OEDOMETER TEST DATA ON COLLAPSIVE BEHAVIOR, TRES MARIAS, AND ADOPTED SOLUTION ASSUMING UPPER LIMIT PREDICTION

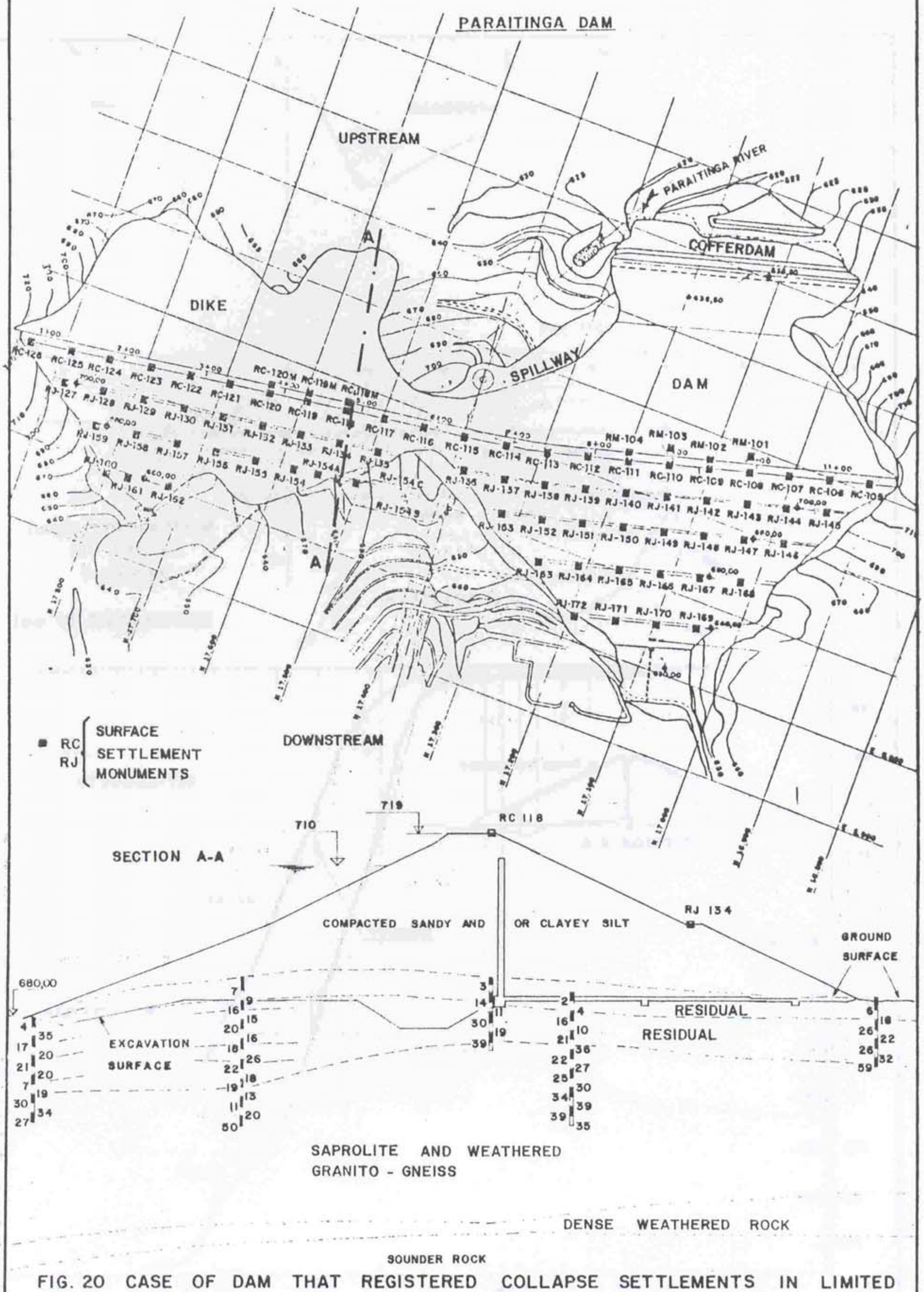


FIG. 20 CASE OF DAM THAT REGISTERED COLLAPSE SETTLEMENTS IN LIMITED DOWNSTREAM AREA

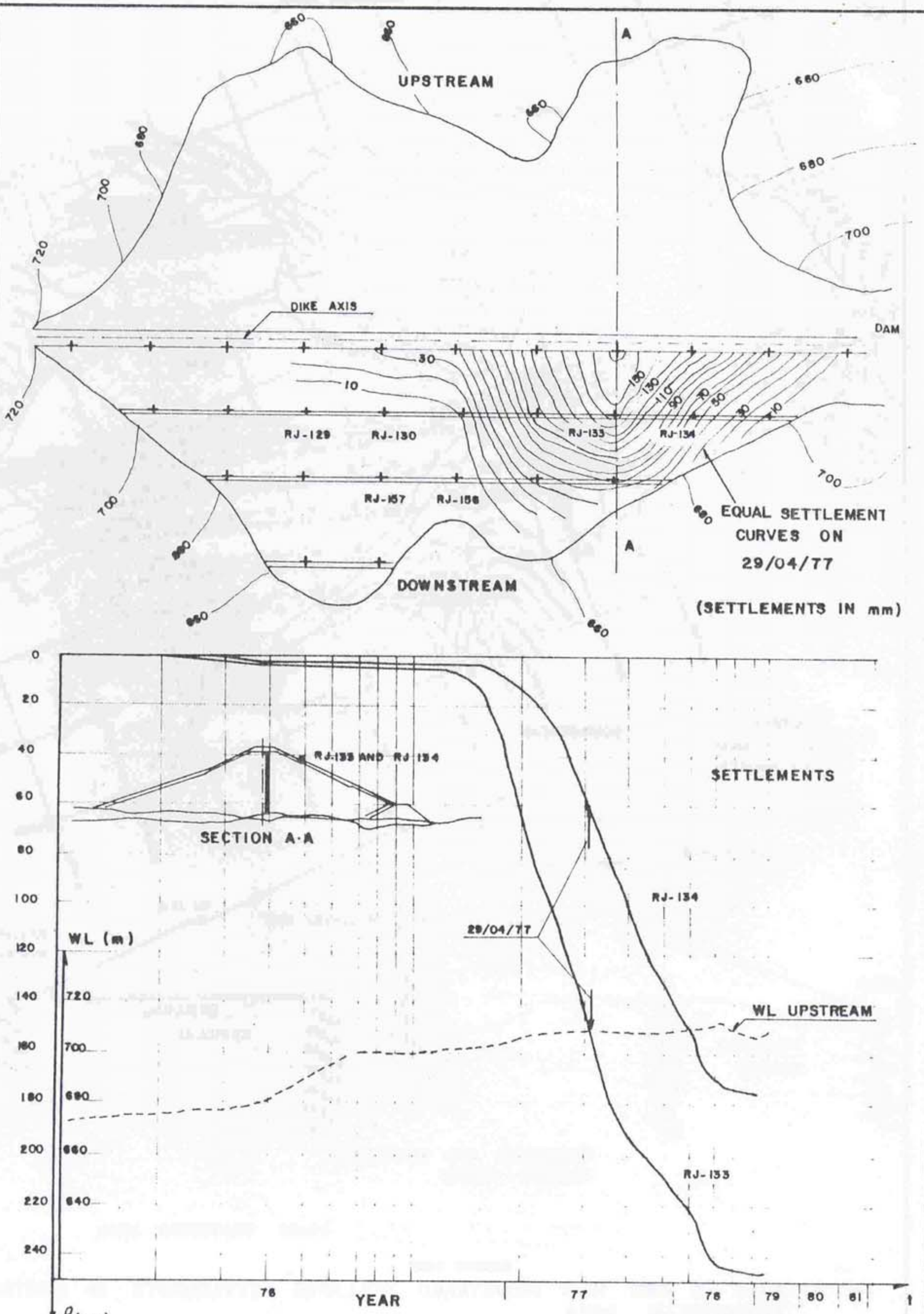


FIG. 21 DETAILS ON FIG. 20. COLLAPSE AND SURFACE SETTLEMENT DISH THAT CAUSED MINOR SURFACE CRACKING