there are few publications (Banerjee et al, 1979), (Evans, 1987).

Liquefaction of soils during earthquake is the first cause of concern. The Committee on Earthquake Engineering of the National Research Council, U.S.A. has published a report (National Research Council, 1985) reviewing the state of knowledge, the causes and effects of liquefaction of soils during earthquakes and documenting the state of the art of analysis for safety from liquefaction. This report reflects the consensus of specialist from the United States, Japan, Canada and the United Kingdom. It is a complete reference on the subject.

When there is no threat of liquefaction or severe loss of shear strength under seismic shaking the relatively simpler method proposed by N.N. Newark is still used (Hynes-Griffin and Franklin, 1984),(Hendron, 1985).

In dynamic analysis it is necessary to have the value of the shear modulus, G. There are many papers on this subject, with determinations by both laboratory tests, (Seed et al, 1984), (Stokoe II, 1985), (Peiji Yu and Richart, 1984), (Chung et al, 1984) and field tests (Sykora and Koerter II, 1988), (Hiltunen and Woods, 1988). The information is ample for sands and meager for coarse materials (Bolognesi et al, 1988), (Stokoe et al, 1988). In a few publications the dynamic properties of the constituting materials have been computed from the actual response of the dam as a prototype (Seed, 1980), (Romo and Villarraga, 1988).

Dobry (Dobry et al, 1982), (Ladd et al, 1989) consider that a cyclic strain approach to the problem of predicting pore pressure buildup has significant advantages over the cyclic stress approach proposed by Seed. Dobry (Dobry et al, 1984) had also presented a new approach to the liquefaction evaluation of earth dams.

Castro and Poulos (Castro et al, 1982), (Poulos, 1981) have introduced the steady state of deformation as an amplification of Casagrande's critical void ratio and developed a method of liquefaction analysis (Poulos et al, 1985). It stands as an alternative to the Seed-Lee-Idriss' method. They have published a re-examination of slide of Lower San Fernando dam based on their ideas (Castro et al, 1985). They are the main contributors to the knowledge of the undrained steady state of sands, including mine tailings. Again the information is very limited for gravels (Bolognesi and Micucci, 1987), (Donaghe and Torrey III, 1985).

Particularly difficult to analyze are dams and foundations made of gravels with cobbles and boulders. Values of D_{50} of the order of 30 mm and D_{max} of the order of 50 mm have been reported (Noguera, 1987). Very few results are available of the proper modeling of the laboratory test specimens of coarse grained soils (Siddigi, 1987), (Banerjee et al, 1979). The same applies to the results of field determinations comparable to the well known SPT, CPT and PMT tests for sands. The use of the Bècker hammer drill has been proposed (Harder and Seed, 1986) as well as special penetration tests (Giuliani et al, 1984). Shear wave velocity measures as an indirect method to determine field densities requires additional information (Bolognesi et al, 1988).

The already mentioned report of the National Research Council, U.S.A., (National Research Council, 1985), contains a chapter on improvement of liquefiable soil foundations conditions. Practically all the available methods are evaluated. For the foundation of dams the most commonly used methods are blasting, densification by vibration and dynamic compaction. Comparisons between conditions before and after the treatment are available for sand foundations (Moreno et al, 1983), (Mitchell and Solymar, 1984), (Harder et al, 1984). For the gravelly shells of dams chimney drains are recommended to control seismic pore pressures (Bolognesi, 1984, 1987),Figure N° 3, because of the large value of the coefficient of consolidation of clean gravels. Blanket drains, toe drains and collector systems are also incorporated.

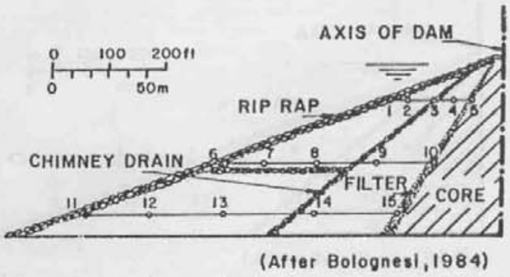


Figure 3. Chimney drain to control seismic pore pressures Cerro Pelado Dam, Cordoba, Argentina.

Because both dynamic analysis and field measurements indicate that very frequently accelerations are multiplied at the crest of the dam, undesirable large deformations of the upper part of the outer slopes can be reduced by flattening them (Bolognesi, 1981), (Seed et al, 1985).

New data and new analysis have been presented of the Infiernillo Dam, Mexico, including the effects of the September 1985 seism (Romo and Villarraga, 1988). high quality systematic measurements since construction (1962-1964) and the use of corresponding refined methods of evaluation furnish important information on the effect of earthquakes on rock fill dams.

5. LESSONS OF ADJUSTMENTS TO TROPICAL SAPROLITES AND LATERITES

by V.F.B. de Mello

5.1. Introduction

In dam engineering the adjustment from region to region is always strongly influenced by many factors beyond the geological, such as meteorology, hydrology, construction plant and logistics, socio-economic, contractual, and so on; in fact, for instance, much of Brazil's dam engineering seems hitherto more influenced by the exaggerated hydrology, affecting cofferdams, closures, rock excavations, schedules, etc., than by direct geotechnical concerns. However, the aims of this Conference set those factors aside. To Geotechnical Engineering, the adjustments between different siting conditions include both foundations and construction materials, the former understandably more influential. Finally, in tropical saprolites and laterites, it is difficult and senseless to separate the conjugate activities of geology, engineering geology, soil mechanics, and rock mechanics, since the transitioning is continuous, and the problems faced bifurcate between those of discontinuities within competent media, and those of less competent continua. No soil mechanics can be compartmentalized and extracted out of the context, of the engineering-geological containment and of the bounds of rock mechanics horizons of cracked and weathered rocks. Recognizedly the interwining is universal: we merely emphasize that it is more pervading and extensive in deeply weathered tropical soils transitioning through saprolites to sound impervious rock.

In a rough historical subdivision one might say that the engineering of dam projects went through three phases: prior to 1948 the overall project concept was dominant, and the subordinate dam design followed empirical rules; between 1948 and 1968, the dam design was generally faced as an essentially independent, dominant unit (especially when embankment dams were chosen), and the embankment dam design became dominated by idealized soil mechanics theorization; from 1968 to date the overall project concept reassumed dominance, with a significant interplay of construction plant and logistics, the search for optimized use of every possible material all the way from big-size sound rock to highly plastic clays, and the broad spectrum of alternatives offered by the much developed industries of equipments and materials. Together with the broader scope multi-disciplinary vision, the recent geomechanical trend has emphasized the problems of heterogeneities, probabilities, questionably theorizable foundations, "umbrella solutions", exponentially increased costs of risks, and concentration on investigations of unfavourable features, but preference for dominating the feature rather than analytically studying safe coexistance with it.

One may envisage that when embankment dam engineering of the postwar period began in much of Latin America, there was much interplay between several key factors: (1) the recent M.Sc. soil mechanicians back from pioneer Harvard/M.I.T./ Urbana schools, with courses concentrated on analytic fundamentals, without chapters on professional design, foundations, dams; (2) a direct consulting influence of such figures as Terzaghi and Casagrande, with no intermediate échelon; (3) the practising professionals, who would fill the gap of theory-to-practice experience, bound to rule-of-thumb pre-geotechnique routines, unable to dialogue with either (1) or (2); (4) the need to adjust (a) academic idealizations of Boston Blue Clay, and Ottawa sand, to very different local real soils, and (b) the analytic fundamentals to complex professional syntheses; (5) the dearth of equipment and techniques for soil investigation and testing, as well as of research-development institutions intent at supporting regionalization; (6) the advent of wartime heavy earthmoving equipment, alongside with the relative collapse of the hydraulic fill dam construction, earlier practised in many dams; (7) the immense advantage of a cauldron of schools of design and construction practices imported from all over the world.

The chronologically natural sequence was to have concentrated first on the problems of the built embankment section, both because foundation problems were mostly obviated by chosen siting, and because the designed and constructed embankments were amenable to the research-type laboratory testing, and idealized theorizations emanated from the Universities.

Experience with dams demonstrates that one may subdivide the analysis of their problems and solutions into three fundamental phases; those of: (a) investigations, design-construction decisions, specifications, inspection; (b) short and medium-term monitoring, corrective and complementary measures; (c) long-term problems and design aims against long-term risk trends and events. Obviously for each successive dam achieved, the lessons from any and all of the three phases are cycled back into the first phase of subsequent dams; therefore phase (b) should gradually reduce to the psychological and benefit/cost acceptance of manageable surprises. Meanwhile with regard to (c) a regionally indispensable decision has to be the attempt to engineer the features of (a) and (b) inasfar as possible, so as to preset the dam's features to conditions worse than anticipatable for the future and, moreover, to employ designed-constructed features that should only benefit from the passage of time. Nothing stays constant with time, things either deteriorate or improve, however little: all technical, sociological, and general considerations lead us to the design recommendation that in dams we can only rest content if long-term trends are engineered for non-deteriorating tendencies. In the following itemized summary of present design beliefs, the observational method Bayesian input recycled from phases (b) and (c) is discussed as part of the state-of-the-art of phase (a).

It escapes the possibility or intent of the present summary to attribute the developments to specific sources and authors, local or foreign, whereupon apologies are advanced to those not specifically cited. Many of the most important ideas sprouted simultaneously under independent clinching thinking, whether or not fertilized subconsciously by prior interactions. It is a fact, however, that many developments that were in use and published locally came to be attributed to foreign consultants who learnt of them in contacts with local long-standing practice, and published them in dominant English language vehicles, with an understandable omission of divulging the authorship of a practice taken for granted as already consensual. Some examples were, for instance, the decomposition of grain-size curves to check against non self-filtering gap-grading, the abolishment of dense compaction of chimney filter-transition drains, the erroneous or dispensable design requirement of a filter-transition between core and rockfill or rip-rap on the upstream side, the use of head-on contact between clay core and concrete, the higher than imagined phreatic and high-proportion head loss in the smallest final seepage path across the higher-dam cores in approaching the downstream filter, and so on.

In the following presentation we attempt to subdivide into four main items 5.2. Compacted materials, 5.3. Interface problems, 5.4. Foundation treatments, 5.5. Misbehaviours and failures, and in each such item submit, with profound apologies, terse comparisons of so-called "Erstwhile and directly imported thoughts" vs "Progressively adjusted and Presently presumed", and although all problems continually interact, in item 5.2. we try to observe as much as possible a professional sequence without formal subdivision (A) Classification and pre-bid design, (B) Specifications, tie of design intentions with as-abuilding prototype (C) Inspection and observation adjustments (D) Prototype behavior analyses reflecting on design preconceptions. Comprehensibly, 5.2. is taken in a sequence of subitems 5.2.1. Core, 5.2.2. Filters-transitions-drains, 5.2.3. US shell, 5.2.4. DS shell, 5.2.5. Random.

5.2 Compacted materials, design cross-sections, specifications, inspection

Deep horizons of unsaturated "red porous silty clays" on the abutments of the early lower dams (25 to 50 m high) automatically led to facile use of Standard Proctor-compacted "idealized homogenous clay-dam sections". Terzaghi early (1947) introduced the narrow vertical chimney sand filter drain slightly downstream of the axis, connected with subhorizontal drainage under the downstream zone. Concerns, tests, analyses, observed behaviors, lessons, and solutions gradually evolved very much in the 40 years, calling for a succinct presentation: merely for convenience this will be based on a presumed optimized single-material cross-section, with different optimized functional zones. On such a background the different situations, both chronological and of geotechnical and professional preferences, are more easily discussed. The fundamental principle is to focus on the desired or tolerable functional zoning, and, thereupon, obviously to approve any condition equal to, or better than, the one used for the reference section.

5.2.1. Impervious core materials

Erstwhile and as successively directly imported

Regarding type of material, dominating trust in plasticity chart classification. Search for CL-CH, high enough Ip for "tough, non-erodible" (subjective), presumed "plastic" compacted material. "Red porous soils" of about $30 < \omega_L < 60$ %, slightly above A-line, sought with a compulsion that in extreme cases led to scraping together 20cm topsoil up to 15 km from dam. Ban on MH-ML associated with cracking, erodibility potentials. "Residual soil" considered unfavorable, presumed high $u_{constr.}$ Grains coarser than sand, gravelly soils, not testable in 1.4" lab. specimens, suspect, or tested via fines as matrix: soils with more than ≈ 25 % cobbles (≈ 6-20 cm) anathema, and/or giving Proctor $\gamma_d >$ 2.1t/m3 questioned. "Clays" of Proctor peaks $\gamma_d < 1.6$ t/m3, $\omega_{opt} > 20$ % questioned as weak and compressible (end-of-construction instability). Keyword "clay" required perforce. No hint of acceptability criterion via compacted permeabilities (e.g. $k < 10^{-5}$ cm/sec.), nor of broad well-graded gravel-sand-silt-"fines" materials (with Standard Proctor peaks ≈ 2.2; 8 %) proving satisfactory. Materials of required excavations mostly wasted, irrespective of sources, qualities, constr. logistics. Saprolites set aside a priori, unknown therefore preconceived bad.

In the 1970's a single additional imported selective criterion summarized under so-called "dispersive clays" with concomitant index tests. Question initially posed as anathema, later quoted as perfectly acceptable, with design provision of fully intercepting filter (which was, in local practice, sine-qua-non earliest dominant design feature anyhow).

Conventional predesign tests, oedometer, permeability, triaxials UU, CU, few CD, as Proctor compacted. Consulting mentors disregarded (and/or distrusted) local test results, when they began to be available (c. 1954). Routine "faithfull" uses of these tests merely "settlement analyses" (ignorantly, but conservatively, taken for post-construction crest settlement camber and freeboard), cross-dam seepage flows (never resulted consequential), and slope stabilities (this single concern constituting > 95 % of embankment dam design and worry, exclusively attended by flattening slopes).

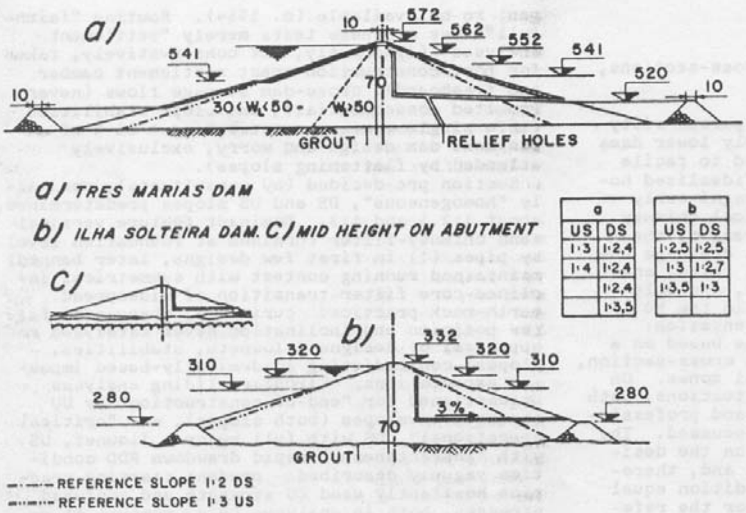
Section pre-decided (by consultants), nominally "homogeneous", DS and US slopes predetermined, about 1:2.5 and 1:3. Dominant feature vertical sand chimney-filter (drained at foundation level by pipes (!) in first few designs, later banned) maintained running contest with symmetrical inclined-core filter-transition of widespread earth-rock practice: curiously, changes of filter position and inclination never catalysed reappraisal of designs (flownets, stabilities, slopes) contradicting academically-based imposing expectations. Circular sliding analyses unquestioned for "end-of-construction" by UU strength envelopes (both slopes), and "critical operational", DS with full reserv. flownet, US with "instantaneous" rapid drawdown RDD condition vaguely described: prudent starting practice hesitantly used CU strength and confused stresses, both in analyses (e.g. removal of reserv. US face hydrostatic triangle). Rarely, if ever, were results of these concerned local analyses either discussed or considered for design changes: Consultants' dominant concerns were high $u_{constr.}$ (cf. USBR prototype u monitoring of the time).

Note in passing (cf. Fig. 4a) that some important cases avoided incorporation of cofferdams, and even required filling the intermediate area up to cofferdam crests (with "random" traffic-compacted soils, etc) "to reduce the effective height of dam", which seemed strange considering the routine adoption of similar constant slopes irrespective of heights. A concomitant practice, presumed judicious but clearly undefendable, situated the subhorizontal DS filter-drainage blanket also above DS cofferdam crest; or even max. maximorum DS water level (cf. Fig. 4b).

Around 1960 the concept of obligatory "zero cohesion" was introduced with deep consequences, theoretical and professional. The questioning (cf. Taylor 1948) of cohesion in "conventional" clays as possibly "not being permanent" in long-term, was well known, but its entry into effective-stress analyses even for end of construction stability analyses brought added confusion in already confused design practices; field experience dictated rejection of flattening slopes of upper shallow sections.

In the 1952-60 period strong influences from Imperial College implanted effective stress analyses: for end-of-construction, using the constant $\bar{B} = u/\gamma_z$ hypothesis which greatly hurt economy in shallower dams; for the RDD, an additional confusing analysis, that resulted independent of dominant drainage design features and used the Skempton-Bishop A, B pore pressure coefficients, with assumed constant and very conservative values. The perplexities and quests were only local, since the Consultants took no notice of either the publications or the resulting design analyses (see development further).

Construction specs, extracted directly from USBR case-history records, "method specifications". Site apprenticeships discovered that U.S. clay inspection testing still using prin-



a		b	
US	DS	US	DS
1:3	1:2,4	1:2,5	1:2,5
1:4	1:2,4	1:3	1:2,7
	1:2,4	1:3,5	1:3
	1:3,5		

EXAMPLES OF SPENDTHRIFT COFFERDAMS AND ERRONEOUS CONCEPT ON DRAINAGE BLANKET

TYPICAL HOMOGENEOUS COMPACTED CLAY SECTIONS (C.1960)

Figure 4

cipally Proctor needle, index of satisfactory compaction (including, especially, compaction moistures, specified in + x % to - y % differences around compaction test optima) with penetration resistances higher than, say, 20 kg/cm2. Reference standards imported (a) Standard Proctor (Normal), PN (b) Harvard Miniature, HM (attempts, e.g. Tres Marias Dam, c. 1958 resulting useless) (c)Modified Proctor, MP, e.g. Volta Grande Dam, etc., c. 1969. Starting reqd. 96 % compaction, PC %, per layer, no mention whether min. or aver. Locally interpreted as rejection criterion, generated intense development of statistical controls, for homogeneity, minimizing rejection-recompaction traumas, achievement of aver. considered logical design aim. In situ dens. testing by sand-cone method imported from U.S., and from U.K. the speedy-moisture tester, rapidly discarded because of gross errors, in clays, and generally insufficient precision.

From 1958-86, with no correlation with increasing dam H or different observed settlements related to soils, specs. incomprehensibly moved up (e.g. 98 %,100 %, 102 % PC), accompanying not rational needs but achievements of heavier equipment and reduced lift thicknesses. Unquestioned excellent inspections and performances have been losing out to diffused published academic concerns, and inexperience of rapid turn-over of junior field engineers: increased wasted costs.

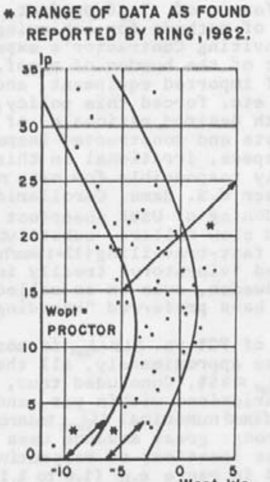
Minor sector attempted introducing MP specs., reducing reqd. PC % (e.g. to 94 %, 96 %) but retaining same form and numbers for moisture range specs. Suggestion diligently investigated by local geotechnicians, understandably concluding strongly against.

In 1958 imported Hilf method for rapid moisture control, millions of inspection test data, and consequent developments into Hilf-Proctor concept tests and valuable practical improvements (see further).

Imported firstly (c. 1954) Casagrande piezometer and USBR cross-arm settl. device. Rejection of the former for Uconstr. easily argued (c. 1956). Imported Mayhak electrical piezometers . (c. 1957) and London USBR double-tube piezometers (c. 1958) to demonstrate intrinsic starting error of two-way flow cell. Imported (c. 1970) the Hall pneumatic cell.

Inclinometer (e.g. Slope Indicator) introduced c. 1970, repeatedly used, almost generally with no useful indications, only significant added expenses of installation and, principally, as with all instrumentation, long-term efforts of zero benefit/cost ratio. Recently (c. 1984) introducing horizontal profilemetering (e.g. BRE etc.).

Regarding design behavior and analyses, one regrettably notes the net impression that no profitable geotechnical design indications were clearly imported post-1965. Apparently justifiable that almost all publications pertained to (a) case-history descriptions without extraction of theses (b) case-histories reporting unusual or undesired features, similary unproductive for generalization (c) academic proposals for eureka analyses, without backup against practical experience (d) self-justifying analyses of single cases to back a hypothesis (e) papers on how given case essentially matched predictions, i.e. demonstrating preference for falling-in-line as against the difficulty and courage to postulate revision (f) etc. Noticeable exception is the clear concept of core compressibility silo-effect hang-up; especially in wet-compacted earth-rock dams on shifting from



DATA FROM CLAYEY RESIDUAL SOILS. COMPARISON OF WATER CONTENTS AT OPTIMUM PROCTOR COMPACTION, WITH RESPECTIVE PLASTIC LIMIT.

Figure 5

dumped to compacted rockfills (Scandinavia): and very preliminary concepts of hydraulic fracturing, cracking, segregation, self-healing of cracks, etc..

Never have cases been presented discussing alternate design choices and quantifiable accept-reject criteria of how the final design preference was reached, a point which is the single priority need of every geotechnical dam engineer.

Progressively adjusted and now presumed-adopted

At start, light excavation plant generally maintained borrow pits in upper clayey horizons SPT < 10, unsaturated, easily compactable, of "granular behavior" until handled and compacted. Disintegratable soft rocks reduced to powder and then compacted (e.g. Tres Marias, 1959). Since beginning, however, even when correcting tests to avoid any drying or reuse of same portion, no direct significance was found from conventional classifications, either grainsize or Plasticity Chart. Besides questioning significance and reproducibility of destructured-deflocculated-plasticized grainsize and plasticity tests in residual soils and millions-years-aged tropical sediments, rapidly discovered erroneous keyword names of index clasification: for instance, "high plasticity" (high Ip) clays mostly produced toughest, friable non-plastic compacted materials (cf. Fig 5, from de Mello 1973, '80, distinguishing between essence of plastic nature and plastic state, showing that ω_{opt} only reached conventional plastic state $> \omega_p$ in range of $5 < Ip < 22$). Slightly-clayed fine sands from totally weathered eolic sandstones gave ideal impervious compacted product, resistant, incompressible, non-shrinking, although also rigid. Because of non-pulverizing field plant logistics, the slightly tougher CH borrows $7 <$ SPT < 15 produced some questioned "core zones", less impervious than the silty MH selected for corresponding shells and similarly compacted.

Classification directly by compaction test position on locus of peaks found preferable, despite many cases of great distinction between lab and field compactions, through widely different nucleations and disintegrations at play. Sometimes necessary to check shrinkage limit void ratio vs. compacted void ratio, problem of deep drying shrinkage cracking and accompanying dangerous reduction of transverse $\sigma'_2 = K'_0 \sigma_v$ condition diametrically opposite to beneficial compaction OCR which increases important residual K'_0. Since earliest cases (1953-8) because of scoldings by consultants on construction inspections, lab-tested for (a) swelling potentiality for sealing cracks (b) minimum overburden consol. press. for "rewelding adherence" on clay-rock or clay-clay slickened unscarified planes.

Residual soils of rocks containing widely different grain mineralogies much preferred, problem cases being residuals and saprolites of some homogeneous sedimentary rocks. Appearance of "excess mica" problem, shining on surfaces of compacted lifts, found much overrated in granito-gneisses and even some schists-phyllites: rare bonafide % mica test data, quoted tolerances in specs. (e.g. <5%) generally bluff. Very high ϕ'_{res} ($> 38°$) tested (Vicksburg 1975) in alleged very micaceous gneiss-saprolite (Paraibuna Dam): result predicted as logical, by dominance of very angular quartz grains.

Rapidly transparent dichotomy of theory-practice in disciple-consultant attitudes (c > 1960) generated intense secret queries and efforts toward logical compatibilization.

"Dispersive clays" much searched by tests, mostly not found, or yielded contradictions: oxidized ferruginous inactive tropical clays seem justifiably exempt. Opposing camps still debating in the fertile confusions generated, with interplay of topics persisting subjective or inevitably difficult to quantify, type of material, cracking, erosion, filters, etc.. Problem of "dispersitivity" of "clays" (or "fines"?) is presently suffering absurdly from having been embraced by geotechnicians on dams, under great inevitable dispersions and confusions on micron-size behaviors at zero stresses: even geothechnicians unprepared with deep intuitions on colloid-chemical tested behavior trends, and colloid-chemical theoreticians stumbling on near-solid state behaviors controlled by dense physical active and inert grain-structure contacts, become stifled in face of the presumed multi-boundary-interdisciplinary problem, if it exists at all. Meanwhile ignorance invited standardization-labelling by the unprepared, and offers a fertile field for conjectures around spotty irrefutable observations at surface (at zero stresses). Is not the respected requirement, to avoid any significant zone of the dam trending toward zero stress conditions?

Laterite concretions, or cobbles-gravels, or rock fragments, found to help core compaction and behavior (mechanical stabilization in wet conditions $\omega_{comp} > 1.15\ \omega_{opt}$ up to % content (e.g. < 30%) that doesn't impair impervious matrix). Powerful excavation plant (loaders etc.) permits exploring pits deeper than SPT ≈ 50, with in situ γ_d higher than compaction γ_{dmax}: compaction becomes misnomer, spreading and compaction plant representing partial remolding, some disintegration some pressed contacts between high-nominal-preconsolidation chunks. Sometimes imperceptibly deepen below saturated horizon, requiring care on u_{constr} generation. Presently (c > 1975) insist on borrow investigations checking in situ percent saturation, and compare in situ vs. fill compacted dry density to distinguish between compaction and undesirable remolding overcompacting slickensiding.

Weathered rock transitions similarly handled, including ripper as necessary, and principle of minimizing disintegration beyond minimum need obtaining impervious matrix. Rippability investigated, SPT borings and published geophysical indices proven easily misleading: (a) greatly increased excavation energies available (b) depth effect higher SPTs eliminated by wide pit excavation (c) ripping much conditioned by tooth-grip on released joint planes, and minimal delay for release effects.

Standard Proctor, PN, preeminent as reference for PC% construction specs.. MP introduced undesired confusion, no benefit, only detriment from duplicity of indices, especially as used, lacking adjustments and comparisons between each other and with field reality. So-called wet-compaction on MP may be dry-compaction on PN; adjustment for lower energy peak (e.g. 94% MP $\approx$ 100% PN) would require shift of ω_{opt}(e.g. ω_{opt} MP$\approx 0.95\omega_{opt}$ PN). Fatter clays unable to absorb the much higher MP applied energies: when drier, obliged for trafficability bearing capacity, problems of higher compaction gradient across lift, when wet, much over-compaction slickensiding, especially dangerous with laminar clay-minerals. In basalt clays needed to restrict equipment weights, pressures, energies(including hauling) below PN to decrease below-surface laminations. Thousands of data from inspection tests in many dams, repeatedly suggest better statistical match of field compaction curves (slightly flatter) with PN peaks.

Important to distinguish between impact applied energy in rigid confining mould, four times higher in MP than in PN, and net hysteresis absorbed energy in field, in cyclic loading-unloading deformations of successive passes, dominantly including shear deformations, much more so under heavier equipment passes: the difference between lab index test and field behavior grows rapidly with increasing pressures, energies, impacts etc. Categorically reject MP referencing in silty-clayed materials, in saprolites etc. of grains subject to crushing and declustering, etc. Note that in clayey unsaturated residuals and saprolites, entire history of field behavior, from borrow excavation, construction, through long-term operation, is controlled by soil clusters, all the more so as more powerful equipment has advanced into facile use of denser (SPT > 50 etc.) horizons. Lab tests on powdered soil growingly lost any correlation with field-extracted behavior parameters. Regarding economics of compaction of multimillions of cubic meters, we lack crucial data on dimishing returns of increasing applied energies (construction cost) as reflected in significant endproduct behavior and design (benefit) parameters: the diminishing returns on the crude lone index parameters of dry density and compaction pore pressures and suctions have much data, but scattered undigested.

Regional problems led to rejection of U.S. "method specifications" in favour of "end-product specs with suggestion of methods for achieving aims", admitting and inviting Contractor's experience at his risk-cost of the burden of proof. Very varied origins of imported equipment, and importing difficulties etc. forced this policy, which also concurred with desired rationale of link between design tests and constructed-inspected realities; method specs, irrational in this matter, suspected partly responsible for many reported failures of lesser U.S. dams. Corollaries resulted as rapid abandoning of USBR sheepfoot and 50-ton rubber-tyred slow rollers, substituted by surprisingly better fast-travelling(12-15km/hr) short-pad self propelled "vibratory" (really impacting) rollers from Sweden, even in so-called fat clays which should have preferred "kneading compaction".

Dimensionless graphs of PC% vs. $\Delta\omega/\omega_{opt}$ in most soils tend to superpose approximately, all the way from $\omega_{opt}\approx 8\%$ to $\omega_{opt}\approx 45\%$. Concluded thus, that in inevitable variations within pit, and from dam to dam, specs of fixed numerical $\Delta\omega_{comp}$ tolerances are conceptually wrong: great advance uses specs of moisture range as fractions of respective ω_{opt}. Wet compactions used in range e.g. (1.0 to 1.1) ω_{opt}, and dry (0.92 to 1.0)ω_{opt}. "fat" basalt clays ($\omega_{opt}\approx 40\%$) have done well, with rock fragments, up to 1.15 ω_{opt} (Salto Santiago 55m saddle dam, c. 1978).

Harvard Miniature reference compaction test abandoned c. 1960, difficulty of jointly adjusting peaks of γ_d and ω_{opt} with field compaction curves (Tres Marias, Paranoa, Santa Branca dams). Regarding valid lab research contentions of importance of different "structures" of compacted clays (c. 1957-'9) two objections arose: manifold differences encontered in behaviors as computed from any-all lab tests and prototype behaviors (better), making different tests a minor second-order concern: also as emphasized (e.g. de Mello, 1980) field compaction involves cycles of bearing capacity compressions, whereas any lab compactions with rigid moulds are quite different, confined. Matching water contents and compaction-precompression dry densities in lab and fields, is no more than a first-order index.

Construction logistics shows that lift thickness spec must be used as a rejection criterion maximum, as spread, and definitely not as compacted, since reducing spread lift is part of construction routine. Attempts to increase lift thickness (e.g. above 25 or 20 cm max. spread) gave high PC% gradients top-to-bottom of layer (e.g.ΔPC $\approx$ 2-5%): multitudinous statistical data. Drier and higher PC% work gives greater ΔPC gradient. In soils with coarse contents, recommend use of MP-size adjusted control tests (e.g. Sao Simao dam, c. 1973, etc.).

In design analyses started with end-of-construction stability by UU total stress. Parallel analyses using pore pressure tests always conceive separating "preparation, preshear", phase and shearing phase, both of specimen and of limit equilibrium analysis: preparation phase handled either by anisotropic consolidation or by effective stress state deduction. However, for shearing phase, reasoning on impossibility of predicting rate of shear, and shearing Δu along sliding surface at sliding instant, all design analyses computed two limiting FS values, upper one using assumed drained, and other assuming "undrained envelope" strength expressed as function of "preparation phase" consolidated stress: real FS given as within the range of the two values (cf. details, at end, on sample-specimen-test suggestions corrective of conventional routines of c. 1960-'5)

For both US and DS slopes critical operating conditions introduced as incremental stress trajectories superposed on end-of-construction, and compellingly dominated by critical flownet stable hydrodynamic pore pressures, with compacted specimens subjected to seepage saturation. For DS slope, rapid shearing should be illogical if due to slowly generated flownet: but rapid flow through discontinuities and cracks, and prudence regarding catastrophic eventual DS sliding impose (a) preference for filter-drain positioned avoiding effect to DS mass (cf. de Mello 1977) (b) recognizing risk of rapid shear $\Delta V \rightarrow \Delta u$, though not with seepage saturation. Thereupon for DS slope long-term unstabilization could only be generated by slow incremental changes of flownet and/or of strength (saturation, etc.), thus (almost) validating drained effective stress analyses unless soil subject to structural collapse under stress-strain-time trajectory.

For US slope, critical operational condition introduced via tendency to incremental effective stress trajectory change from stable hydrodynamic, max.reserv. WL flownet ("preparation state" for compacted elements subjected to seepage saturation) to instantaneous transient RDD flownet: instantaneous tendency to change of respective volumes generates corresponding transient u superposed on RDD hydrodynamic flownet Δu. Both flownets compellingly dominated by boundary conditions of section's filter-drainage features. Emphasize that first-order solutions (e.g. Bishop 1952, Lowe & Karafiath 1959 etc.) could suffice, conservatively, in oldfashioned design sections, but far from acceptable in engineered designs employing real advantages of filter-drainage or non-existing filter pressure equalizing features (de Mello 1977, '79). Change of conditions from first to second flownet logically instantaneous, incompressible fluid if-when saturated. Finally, as exemplified in Fig. 6 in representative research tests (cf. Souto Silveira 1969), use of A, B pore pressure coefficients difficult to adjust judiciously or justify.

It is of interest that the meticulous research testing was prompted by questions discussed at Tres Marias Dam, and, despite some questioning that subsequent and present-day refinements could suggest on the 30-yr. old techniques, the basic points of Fig. 6 remain: (a) curved statistical regressions ably reproducing u as $f(\sigma_3, \sigma_1-\sigma_3)$, therefore usable for predicting design u values; (b) no linear behaviours justifiable, the principle of the A, B coefficients having been an idealization requiring adjusted continuous variations. Attempts to predict u using constant A, B coefficients lead to absurd differences.

US slope presumed mostly over-designed: evidence of high reservoirs emptied in few hours due to overtopping failures supports the otherwise difficult-to-confirm postulated behaviour (cf. for ex., Fig. 8 case, mentioned below). It is still a challenging debate, and viable prototype testing is being designed bypassing via other civil eng'g. works: the intent of our report is to stimulate debates which matter much to our staggered economies.

Strength equations adopted have long-since emphasized compaction precompression and curved envelopes for OCR > 1. Neglecting cohesion, using constant $\bar{B}$ coefficient, and using outmoded RDD analyses heavily penalizes long low abutment dike sections: moreover long-term secondary compressions and ferruginous cementations favor true

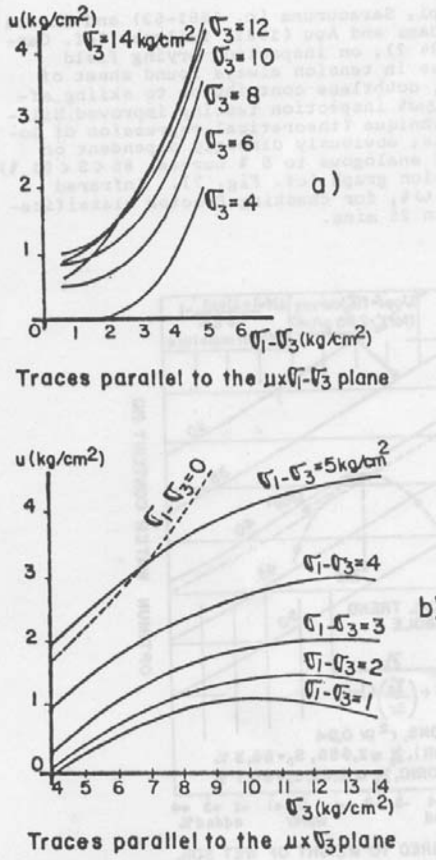
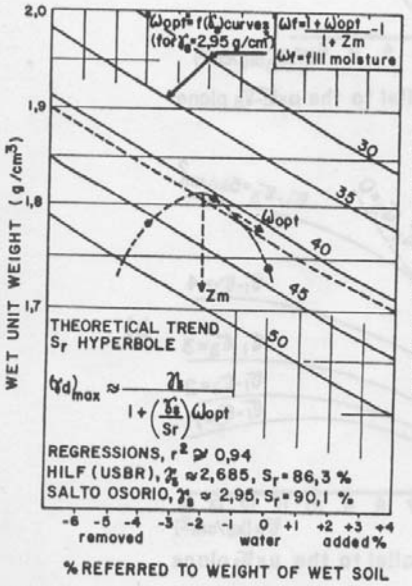


Figure 6

cohesion. Higher PC compactions show up mostly in higher compacted pc values, much dependent on $\Delta\omega\%$, compaction u, and overcompaction. Borrow pit and fill compaction testing are very important, both relying on wet unit wt. (driven cylinder, calibrated nr. of blows for penetration giving first indication of adequate consistency) and on Hilf-Proctor test for $\Delta\omega\%$. In borrow, important to interpret from γ_d and S % whether soil (air pores) will be compacted or merely remolded and shear slickensided. In fill, great importance to overcompaction horizontal slickensiding at 20-40 cm depths, in near saturated condition. Recent inspection specs suggested include trafficability (too dependent on vehicle wts. and tyre pressures), suction measurements (sensitive to tip installations, high dispersions) and portable penetrometer resistance (danger of no indication on slickensiding anisotropy). Recent wet compaction brittle rapid failures explained by high pore pressures (average): emphasize probable erroneous back analyses that forced lowered residual ø' using interpolated u values, when problem lies in higher localized u along shearing plane typical in rapid brittle sliding. In overcompacted Nhangapi, Saracuruna (c. 1961-63) and Itauba (c. 1978) dams and Açu (1981) failures (cf. Carsington 1984 ?), on inspection prying field slickensides in tension always found sheet of free water, doubtless contributor to skiing effect. For $\Delta\omega$% inspection testing improved Hilf-Proctor technique (theoretical regression of locus of peaks, obviously directly dependent on γ_s, suggest analogous to S % curves, $86 < S < 91$ %) and correction graph (cf. Fig. 7). Infrared ovens give ω%, for checking Proctor classification, within 25 mins.



GENERALIZATION AND PRACTICAL IMPROVEMENT OF HILF METHOD FOR COMPACTION CONTROL

Figure 7

Prototype behaviors mostly excellent. In installing Casagrande piezometers about 10 m upstream of completed higher (e.g. 70 m, Tres Marias, 1961) dam vertical filters, confirmed hydraulic fracturing huge losses of drill and pumped-in water. To the contrary, in inclined cores with no hang-up confirmed theoretical prediction (de Mello, 1977) most (≈75 %) head loss in final (≈25 %) flowpath before filter. Some cases of aerobic limonitic concretions recorded in DS drainage blankets when not permanently submerged: present designs prefer such drains under moderate permenent head.

Regarding sampling, testing, and extracting fundamental parameters for computational validating design decisions, attention concentrated on shear strength, deformability, and permeability. Question of utmost importance for design has been the extraction of test parameters for predictions of performance. Design really employs parameters and computations aimed, not at target predictions, but at not crossing the presumed safe-unsafe boundary: that is why designs proceed despite ever-insufficient knowledge and refinement. Terribly important, however, is the commitment, controlling economy, to narrowing the gap between quantified predictions and observed performance, and therefore for seeking a progressive conscious revising school, as was the SHANSEP for natural clays. The publication of test routines in authoritative books caused a great setback to design practice, principal contributing factors being the commercial and prestige inertias of oversize owner and design companies in crystallizing conventions as dogmas, and a cultural error of confusing civil engineering technology-art, the art of opting for the good enough transient and temporary knowledge-solution aimed at avoiding undesired bound of behavior, in comparison with the scientific basic truth to be built upon. Intervening groups, when distant, aggravated the problem by respecting prestige exponentially proportional to the distance, and economy-commitment inversely so.

Of the three principal parameters listed, only the last two have gathered wealths of consistent and significant documentation. Construction-period deformability (settlements) should appear easily tested, but real behaviors have been about 3-7 times more rigid than as extracted from lab. molded specimens, and undisturbed block samples from compacted fill. Propose, for instance, that field sample should be tested in triaxial, but specimens of x % PC being carried to (x + 2), (x + 4), (x + 6) % PC under (σ_v,σ_h) stresses, in order to test under controlled specimen-preparation conditions, for obtaining the behavior at x % PC by back-extrapolation. The undisturbed specimen-preparation should be achieved under cycles of ($\Delta\sigma_v$, $\Delta\sigma_h$), until desired PC % reached: the $\Delta\sigma_h$ estimated approx. through $K'_o\ \sigma'_v$, with $K'_o \rightarrow K'_p$; thereafter σ_v should be reduced to zero at constant volume, with σ_h adjusted as needed. As established over 30 years, the assumptions of undisturbed sample specimens as representing "intact soil element" conditions are definitely unaccepted, and increasingly so in stiffer material, and with insufficient suction to contain strains. Research comparisons needed, abolishing presumed oedometer strain-control, and seeking stress-control testing, clear of disturbances around sample and specimen fabrication conditions.

Construction-period potential unstabilization preferred analysed by total stress and UU tests, but with important proviso of using curved strength envelopes and incremental analyses of ΔFS with rise of fill ΔZ: serious cases of failures have been associated with myopic linear concepts, rapid incremental reductions of FS in final stages having been fatal even with good pore pressure instrumentation. Insist on fallacy of average $u_{const.}$ piezometer observations as indicative of developed u along failure surface at failure instant. Back-analyses questioned, leading to low ϕ' (and ϕ'_{res}) values, presumed because of lower than probable intervening u values: important to recognize non-circular (essentially cycloidal) failure surface, subvertical transitioned into subhorizontal, and frequently inapplicable average strengths because of horizontal weaker plane and significantly different straining (a) from recent top to earlier bottom (b) at low rigid vs. high plastifying confining stresses. Have progressed to preferring brittleness estimation by different slopes of stress-strain curve before vs. after peak (cf. de Mello 1977) in lieu of Brittleness Index proposed by Bishop (ratio of peak to ultimate strengths).

Regarding DS zone long-term stability, emphasize important fully-intercepting inclined chimney filter-drain, avoiding perceptible unstabilizing factor from reservoir flownet. Beneficial effects of secondary compressions, even minor, and of cementations, have not been considered, but must be significant (e.g. inferred from eroded stable 20 m vertical scarp of Euclides da Cunha dam, 1976, when failed by overtopping because of spillway misoperation. (Fig. 8, photo). Great economies foreseeable if/when supported by needed research, especially in shallower dam sections.

Figure 8

Since 1952 attempted testing pseudo-saturated specimens for operational stabilities, firstly using forced seepage tendency to saturate in triaxial conditions. Back pressure saturation (since around 1960) undisputably accepted as boundary engineering solution. Reject arbitrary convention of pseudo-saturation at 100 psi (7 kg/cm2, bars) employing successive $\bar{B}$ checks by $\Delta u/\Delta\sigma_3$ until reaching 100 %. Micro airpores have been evidenced both by much higher (10-15 bars, etc) frequently required for achieving $\bar{B} \approx 100$ %, and by frequent sine qua non need to back-pressure saturate first under low σ_3, before applying desired higher σ_3 (for UU) or σ_3 cons. (for CU). Reflect the tremendous damage to project economics in cases of low reservoir pressures and/or shallow potential failure surfaces, conditions absolutely incapable of being saturated. Much research needed on all triaxial tests (including cyclic etc.) for correction factors referred to conventional fully-saturated tests, in conditions of small (3-8 %) percent air-pores and inevitable time-lags, of development of u, and of transmissions and recordings thereof (in lab specimens, and in prototypes). In stability analyses of any material subject to significant strain-softening (rare) and strain-dilatancy or strain-compression, have found imperative the use of upper-and-lower-bound analyses: one via effective stresses with presumed pore pressures, and the other with anisotropic consolidation to pre-failure potential cause, followed by constant volume quick shear total stress incremental effects.

Homogeneously compacted dams higher than compaction precompression pressures present greatly reduced permeabilities (cf. de Mello 1977) at greater depths, altering idealized flownets. Regarding effective permeabilities, signal the roughly 100-1000 times lower lab permeabilities on powdered-compacted lab specimens compared with field masses of clayey clusters compressed into interface contacts: meanwhile field tests frequently adulterated upwards by suctions, hitherto not included in idealized equations for interpreting flow-time results.

5.2.2. Filters-transitions-drains

Erstwhile and as successively directly imported

The quality of filter materials was always specified merely on the basis of the Bertram-Terzagni criteria for the grainsize at the D_{15F}, which was interpreted to incorporate obligatory upper and lower limits, $D_{15F} < 5D_{85B}$ so as to guarantee stereometric hindrance by the pore size, and $D_{15F} > 5D_{15B}$ intended to guarantee that the seepage velocities in the filter be automatically much lower ($\approx 1/25$, if $k \approx 100\ D_{10}^2$ (Hazen) $\approx 100\ D_{15}^2$) and therefore themselves not subject to movements under the seepage stresses. Compaction requirements were automatically copied from USBR specs of > 70 % RD. The width of tolerance at the bigger grainsizes was much discussed, between requirements of uniform material, parallelism to base-material grainsize, or preference for broader grading (limited by fears of segregations).

Great geological difficulties of procuring desired materials imposed frequent use of crushed rock fines and mixtures with fine sedimented sand (uniform) from weathered widespread eolic sandstones: experienced practical difficulties of perfecting mixing of composite grainsizes from very different grain shapes (angular vs. round) and densities (basalt vs. quartz etc.). Gradually generated own questions, and nominal design answers, both exported to international circles, on: (a) filter criteria critically reexamined; (b) grainsize gap-grading; (c) compactions really preferred and/or necessary; (d) transitioning not merely in seepage erosion, but also in deformabilities and strengths; (e) important need to wash silty-clayey fines from chimney-filter material obviating tendency to accumulate at bottom, clogging abrupt change from subvertical to subhorizontal drainage.

Early designs never discussed drainage capacity (obviously) only affectable by core cracking and by foundation seepages, especially in abutments. First designs used pipes, and sought free-flow condition at base of chimney (later undesired because of aerobic limonite depositions). Later, for some years, trend requiring thick (0.8 - 1.2 m) layer of sand: generated practical discussions, ref. need to spread and compact in single lift, criticized by some Consultants ref. insufficient compaction transmitted to base, submitted as critical to seismic liquefaction (N.B. postulated by ourselves as most improbable because of unsaturation).

Present situations

Experienced gap-gradings in mixing led to index-criteria for quantifying gap-grading in natural materials (cf. de Mello, 1975).

Early recognized that stereometric filtering of coarser representative base grains had to be by representative finer pore sizes of continuous grainsize filter materials, and Brazilian school (A. Silveira, c. 1963 on) generated the quest for probabilistic representations of pore sizes, spreading worldwide quests. Porosimetry of residual soils, with dominant macropores, remains major factor uninvestigated.

International papers recently reported multitudes of filtering, clogging, and washing-through tests, net trend being toward validating Bertram-Terzaghi criteria. Important theoretical and practical limitation of those tests must, however, be emphasized. de Mello always (c. 1958 on) emphasized probabilistic nature of grain arrangements of any frequency distribution curve (e.g. grainsize), and especially extreme-value localized conditions (a fortiori by segregations etc.) as conditioning starts of piping. Fig. 9 plotting Sherard's (1984) test results of points emphasized as being of clear-cut non-failure/failure conditions firstly seem to ratify that the Bertram-Terzaghi criteria of boundary specs should be generally satisfactory. However, the inexorable very broad dispersion of a criterion supported on a single simple index quantification (D_{15F}/d_{85B}) raises statistical-probabilistic suspicion: moreover, the apparent trend of decreasing ratio at failure in coarser materials, although seemingly contrary to intuition, calls for investigation-explanation. Fig. 10 employs the same test data, but using (indiscriminately) a Gumbel extreme-value (cf. de Mello 1977) probability paper of extreme urban flood recurrences, and separating the tests on sands-gravels and those on silts-clays. The trends and regressions seem of interest and impact: the conclusions would derive that Bertram-Terzaghi criteria would incorporate 20 % and 0.5 % extreme-value probabilities of piping failures in sand-gravels, and in silt-clays, respectively. However, in one step further Fig. 11 uses a much broader spectrum of published test results, distinguishing with regard to directions of seepage effective stresses: although all tests were performed on specimens contained in rigid moulds (which is far more favourable than in prototypes subject to shear/bending deformations of the relatively unconfined core-filter interface), and used smooth-rigid core-filter interfaces (greatly reducing probabilistic erosive failures), it is of interest to note in Fig. 11 the significant (although short term) differences between three conditions of flow, vertical downwards (compressive), horizontally, and vertically upwards (expansive).

The topic of filters in prototype conditions seems doubtless benefited by trends to auto-stabilization (cf. de Mello 1977), and thus mostly acceptably solved: however, the geotechnical-probabilistic behavior is irrefutably far from reasonably investigated.

Downstream subhorizontal drainage blankets have been often employing sandwiched coarser layers (expensive) for much increased drainage capacities on semipervious tropical residuals, saprolites, laterites: important internal impervious blanket (cf. de Mello 1977) together with strong impermeability gradient under central highly loaded zone, and with shortened drainage blanket, present obvious design-performance improvement. In some dams the expensive, prudent, DS sandwich-filter-drain has proved quite unnecessary, all the foundation seepage existing within the decreasingly compressed foundation horizon towards dam toe. Must note some design preference for aiming at DS drains operating saturated, avoiding aerobic crystallizations and presumed (difficult, ?, in sequential reasoning) chemical (ferruginous) clogging.

Inclined filters-transitions presently preferred uncompacted, sufficient to maximize light packing rearrangement under pad-vibration and copious watering (pretest principle vs. localized concentrated core-crack flows). Argument regarding potential seismic liquefaction quite absurdly pessimistic, considering very unsaturated filters-transitions. These constitute principal hang-up feature, highly undesired, a fortiori if unnecessarily compacted. Design suggestion easily eliminates hang-up using judicious longitudinal intermittent compressive.

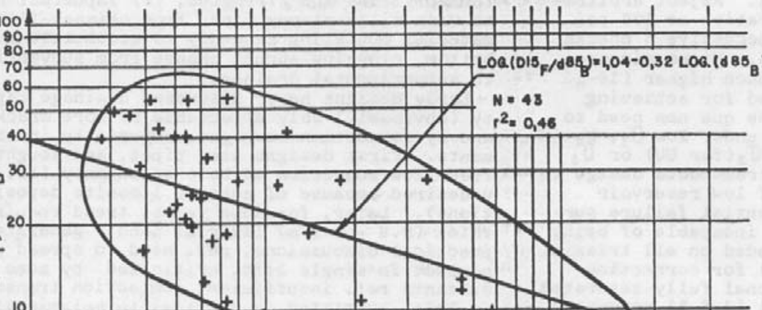


Figure 9

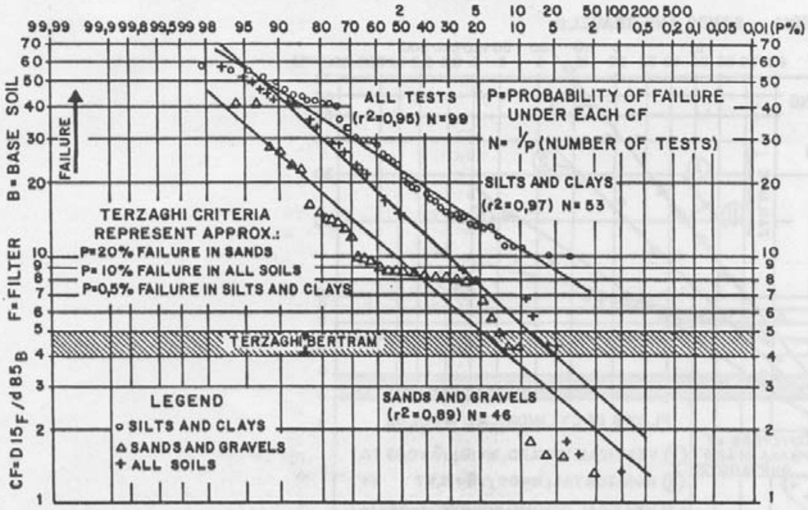


Figure 10

clayey cushions at intermittent height intervals, profiting of three-dimensional strain and seepage redistributions.

Regarding transitions under rip-rap, rapidly concluded limited importance of strict filter requirements, principal functions being uniform stress distributions on embankment slope, so as to avoid erosion-vulnerable zero-stress areas, and wave-buffering by well-interlocked continuous-transitioning grain and pore sizes. Great preference for run-of-quarry single broad-graded rock material, and inside-outward selective segregation obtained by construction procedures, especially in spreading dumped loads into lifts.

5.2.3. US shell

If the US shell is built-up of compacted soil materials, much of the above, 5.2.1. Core, continues applicable, with easily interpreted adjustments on quality of materials, compaction specs, and some preferential distributions in the geometric-geotechnical section. Trends are easily emphasized if one recognizes the compacted earth-rock dam as dominantly preferred, with US shell of compacted rockfill. The conditions sought are of materials and compaction specs maximizing effective stress friction resistance. Because of foundation permeability the base layer as compacted impervious constitutes a favoured blanket, which however must seek imperviousness without impairing strength parameters: these are frequently optimized with SC materials and igneous rock residual soils with angular quartz grains and ferruginous cations, factors proven to have induced very high ø' values. Somewhat drycompacted material preferred, only limitation being avoidance of collapse settlements on first reservoir and soaking, a problem of concern also in cases of rockfills, clean or dirty. US shell offers ideal possibilities of judicious use of layers, lenses, chimneys, etc., interspersed as exiting or non-exiting (cf. de Mello 1977) filte drains for pore pressure equalizations under $u_{const.}$ and/or RDD u. Easy to obviate constr. pe riod instability with dry compaction. Optimized US shell (especially in compacted rockfills) avoids settlements noticeably less than core, avoiding core hang-up.

Experience with compacted rockfills advanced greatly (Mexico, Brazil, Colombia, etc.): principal points include acceptance of dirty rockfills (with provisos ref. fines filling, or not, voids of coarser grain structures without inopor tune redistributions-displacements at first filing), preference for broad-graded multi-contact rock fragments if clean angular and somewhat crushable.

Regarding acceptable quality and grainsizes of material, it's revealing to interpret how the early steps of soil mechanics testing and misplaced faith in its index tests introduced ignorant and inoperative spec routines, reproduced blindly from project to project, merely because of subconscious need for test quantifications. Examples that still affect project costs and ab surd specs and discussions must be cited. In shifting from high-lift dumped rockfills to layer-compacted rockfills, obvious trend was to limit spread layer thickness: well recognized better compaction (yielding smaller deformability) of thinner lifts (Note that rarely, if ever, find recognition of superpositions of benefits of upper compactions to immediately underlying lifts, nor the indispensable discussion of benefit/cost ratios): many, however, intuitively see big oversize rocks with presumed greater friction angles, preferred in outer zones nearer slopes. Important recognition of each lift as inevitably and favorably comprising two sublayers, lower with much coarser rock fragments and voids, thinner-upper with interlocked crushed fine rock and "fines". Given max. lift thickness spec, the natural restriction should be D_{max} not to exceed lift thickness: specs on D_{aver} are clearly academic, inapplicable, incon-

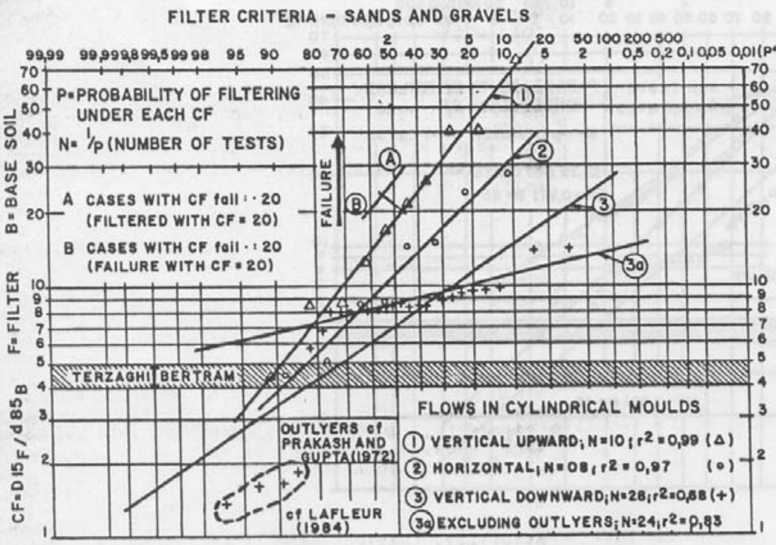


Figure 11

sequent. In limiting the % fines, absurdly resorted to grainsize rather than easily applied, consequent, direct test, e.g. rough observation of rate of disappearance of given water flow poured from water truck, and washing of fines into underlying coarse sublayer. Specs limiting % fines, and these as % passing sieve, expose academic geotechnician quite unaware of rockfill and field realities: the criticism is aggravated if the % fine limit is on # 200 sieve, expensive and easily torn, etc., too minute for significance. Besides exposing engineer who has never seen field operations and projects, also denounce ignorance of experience of early 1950's with boulder-gravel-till shoulders and cores (e.g. Goshenenalp etc.) emphatically proving importance of continuity of grainsize curve, and its shape (e.g. concave upward, minimum pores). With regard to compaction inspection, emphasize importance of intergranular contact crushing, imparting significant compressibility precompression behaviors. Dry density and porosity inspection tests only applicable in comparative terms in same lift, much preferably substituted by measurements of ΔH compressibilities of lift, since the density-porosity indices greatly disperse, affected by the grainsize in test volume and even varied mineralogy and γ_s of distinct rock fragments (in basalts, 2.6 to 3.1 t/m3). Foreseeable great promise for rollers automatically recording geophysical wave-transmission-receipt horizontal moduli.

In upstream-deck rockfill dams, obviously in general way the better compacted conditions reflecting in higher self-weight moduli (less settl.) also tend to improve the really important reservoir-filling face deformations. However, second-order correlation not preserved: using more economical steeper slope tends to increase during-construction self-weight settlements, but face deformations under reservoir pressure postulated as likely to be decreased, which adds technical to economic benefit in comparison with present conventional practice. de Mello (1985) postulates stability (Factors of Guarantee compared with conventional Factors of Safety of slopes of up to 2V:1H (two on one) in strong angular rocks.

Influences of rock quality index tests, wet vs. dry unconfined strength, and Los Angeles values, not yet systematically queried: data used intuitively, some inevitable surprises.

5.2.4. DS shell

Principles affecting the DS shell are mostly similar to those applicable to the US shell, but with overriding advantages, if the fully-intercepting chimney filter-drain, somewhat US inclined, avoids any perceptible effect of reservoir-filling flownet on the unstabilized DS mass. Designs that permit significant interference of onset of reservoir flownet on DS slope unstabilization should be rejected on basis of risk philosophy: DS shell should resist judgments by Factors of Guarantee, in light of catastrophic hypothetical consequences. Tropical soil time effect micro-cementations, uninvestigated but observable in natural conditions (if expansions excluded), all-important in progressively increasing stability. Must note additional benefits such as unsaturation, suction, dilatancy for shallower shear surfaces, etc.; call for bold rejection of conventional stability analyses postulating perfect (back-pressure) saturation (incl. for seismic cyclic tests etc.), zero cohesion, etc. Significantly steepened slopes have been and are usable. UU tests, long-term secondary compressed, unsaturated, are applicable to long-term unstabilization analyses, sliding hypothesis often implying moving into OCR conditions, stress trajectories automatically applying stress increases being questionable.

5.2.5 Random

Various uses have been given to so-called "random" materials in zoned embankments. In principle the acceptance should be of any material spread in moderately thin layers (e.g. 35-50 cm) to be merely "homogeneous", strength, compressibility-deformability and permeability being of no concern. Most uses have considered a presumed "stabilizing weight".

Advances in dam designs over the past 30 years have shown that theoretically no zone can dispense all requirements of fundamental parameters. For instance, DS of fully-intercepting chimney filter, well within center of dam, one could postulate great tolerance of strength and permeability requirements: but not so regarding compressibility. In fact, essentially all random so-called stabilizing berms are inoperative, because they do not apply (soft) weights on the mobilizable shearing surface, and add no stabilization because of incompatible stress-strain mobilization compared with rigid compacted material.

In practice, random zones inevitably become much more expensive than presumed selective spoil areas, no contractor accepting at zero cost a selective spoil deposition, and no inspector tolerating a totally unspecified random placement.

5.3. Interface problems

Principal interface problems have included (a) core foundation contact (b) upstream and downstream shell contacts (c) special considerations of above on abutments (d) interface of compacted-soil/chimmey-filter (e) mid-fill transverse contact slopes, phase construction (f) galleries (g) gravity section transitions.

5.3.1. Core foundation contact

Many core contacts taken to meticulously cleaned to so-called sound groutable rock surface: widths of special sealing treatment have been decreased from about 0.75H (c. 1955) to 0.25H (analogy from narrow-core earthrock dams), fully recognizing geometry quite irrelevant compared with variations of eventual crack widths (to be taken up by grout). Doubtless worst, inoportune, work item and source of discussions, complaints, claims, delays. Slush and broom grouting, later expanded into shotcreting, mostly required and used in great exaggeration, associated with extensive low-pressure area grouting. Must guard carefully against generalizations, especially in light of extreme cases such as Teton (1976) etc.: however, vast majority of non-volcanic, non-karstic (etc.) geologies have comprised non-erodible rock cracks inevitably compressed under higher dam sections. Even assuming core erosion to fill all cracks, core material eventually losable absolutely microscopic, irrelevant.

Debates on rock surface contact concerning alternates of applying surface mortars inmediately before spread of first soil layer, vs. significant anticipation, creating rigidly set hard film. Sterile subjectivities should be promptly unmasked and objectively resolved. Former alternative practically results in an irregular bottom film of soil-cement, better recognized and provided as such, if desired; latter frequently lacks adherence on partially weathered rock surfaces, and mostly results in incompatible rigidity between supporting rock and soft fill load, therefore suffering intense unsticking and cracking.

Great irregularities of cleaned rock surface became inevitable, construction trauma, practically resolved by overdoses of dental and negative-slope concreting, and even generalized pad concreting. In tropical horizons mostly prefer to excavate to geometric grades on dense saproli te to weathered rock (e.g. SPT $\geq$10 + 0.25 (H-30) respecting, and preserving unadulterated, the in timate contacts between the saprolite and its own parent rock. Often elucidative to reason against additional layers x cm thick, and discontinuiti by interpreting in the limit the foundation horizon or th core bottom being H higher or not.

5.3.2. US and DS shell contacts

The US functional requirement becomes limited t strength parameters, particularly as related t sliding along the near horiz. plane. Early requirements, as imported, comprehensibly very se vere because of suspicions on shear strengths of residual soils and saprolites.

Experience and back-reasoning support significant relaxing, although subjective precedents offer continued hindrance. In construction perio stability, tropical saprolites generally prove unquestionable up to steep rockfill slopes (hig dumped rockfill slopes of 1:1.3 to 1.5 cofferdam stand proof), principally because of instant com pression, zero u const., and improbable horizon tal contact critical plane (except with greatly reduced aver. strength) of viable sliding mass. For reservoir operation must begin by emphasizing that much of US foundation behaves under OCR higher strengths.

The DS support-contact function required repeat approx. the above references, and adds preferen ce for perviousness, obviously with piping-erod bility guaranteed as excluded: the gradual perme ability gradient toward DS toe because of trian gular load compression (cf. de Mello 1977) is most favorable. Surface compaction under DS fil ter-drainage blanket should be earnestly set aside, because saprolites reworked and compacted become much more impervious (especially vertical and sealing effect against flowlines exiting in to built filter-drainage blanket is undesired. Almost generally observed localized seeps DS of toe, in holes and/or areas, almost no case wit unstabilizing tendencies except in steeper abut ment slopes.

5.3.3. Abutment support conditions, complementa ry considerations

Unsaturated "porous" abutment supports for dam tions posed deep concerns from the start, parti larly inasfar as theoretical or pragmatic refer ces inexistent. First (shallower) dams satisfi with shallow narrow dozer trench(es) to "intercept continuity of foundation-fill preferential pervious interface". Accompanying then prevale fashion of settlement cracking, first important concern was of prospective large settlements, p sumed big differential settlements of consequen and associated transverse cracking potential. vious acceptance of concept of foundation being functionally equal to or better than overlying (N.B., whenever possible) stumbled on problems sampling-testing-interpreting for field accepta criteria.

Most subsequent solutions/problems occurred through local experience (interacting with Consultants' acceptance), increasingly higher dams and geologic-geographic millions of sq. km. of plateau of unsaturated porous abutments. Incredible wealth of data-experience on SPT vs. σ allow. for pad foundations of highrise bldgs. suggested first indices for foundations acceptability: well-compacted clay fills gave Brazilian SPT values around 12-18, and rough estimate suggests that in situ geologically-aged material of 20-40% lower SPT would behave no worse. Later index on foundation in situ PC% used experience from scores of millions of thoroughly inspected-tested-monitored fills, indicating acceptability of in situ geologically-adged soils of PC% about 4-6% lower than fill.

In fact, however, construction quick settl. surprisingly never led to problems, visible or performance-detected. At Tres Marias Dam (c. 1958) left abutment 9m thick upper horizon of porous red clay (SPT≈2-4, PC≈70%, S_r≈70%), under 55-20m section, predicted and observed quite similar quick settlements, about 1.3m max. Tolerable differentials argued by ourselves as not being more stringent than the published Skempton-McDonald (1956) criteria for cracking wall panels (brick, reasoned more rigid than compacted clay). Acceptance criteria of about 1:100 longitudinal distortions have been used with no further/better backing. In fact, de Mello (1977) queried why abutment settlements should be singled out for concern, when at about 45% height major dams on rock bottoms had upper 55% fills suffering analogous quick internal monitored settls. up to 1-3m. Layered construction, compaction residual stresses, and head/flow loss across minimally thick highly impervious compacted soil presumed to contain justification of apparent no-crack behavior. Publications never qualified tension vs. shear cracking, but locally emphasis was persistent on the important behavioral difference, only the former being critical, only possible in upper parts (low stress zones).

Second concern regarding abutment soils centered on permeabilities. Very high anisotropies, sometimes dominant vertical because of macropores (roots, infiltrations, insects-animals etc.), sometimes dominant horizontal (geologic strata-horizons) differently faced/treated (cf. item 1.4). Worst problems encountered in shallower sections higher up, more matured subsoil, and embankment loading insufficient to compress beyond the "nominal preconsolidation pressure". Under higher embankments the compression/permeability US-DS gradient under trapezoidal loading appears to have constituted a boon: frequent observation of DS flownet contained within subsoil horizon, not reaching subhorizontal filter-drainage blanket.

Problem of collapsivity and consequent settlement-cracking in reservoir first-filling raised by ourselves (cf. Santa Branca Dam c. 1956, Tres Marias Dam c. 1958), continues highly problematic, between extremes of very easily excluded, and so difficult as to require major solutions (e.g. Guri Dam, Venezuela c. 1977) or responsible for inoportune sudden wide cracking and failure. SPT index totally unacceptable because of cementations, unless focussed on difference of SPT in "dry holes" vs. soaked: identification-quantification problem persists partially because of collapse by concomittant Δstress plus soaking. Double-oedometer tests seem too pessimistic, stress-controlled triaxials preferred for collapse estimation. Great differences derive from differentiated depositions, cementations, and rainfall infitration climatology. Simplified indices from one region (e.g. USBR) cannot be transplanted to other conditions.

Problem of very low strength parameters in back-pressure saturated specimens belatedly (ref. e.g. Tres Marias) raised great concerns on operational slope stability (cf. Promissao Dam etc., c. 1974). Test and interpretation procedures intensely debated, and still calling for better elucidations. Prototype behavior surprisingly heedless of theoretically-postulated danger: explanations presumed to include very gradual tendency to saturate without achieving it, significant anisotropy and OCR of foundation before flownet, test and failure criterion interpretations etc.

5.3.4. Interface of compacted-soil/chimney-filter

Considering the pioneering condition of the Terzaghi vertical chimney filter-drain, narrow beyond all precedents (e.g. 0.6 to 1.0 m), comprehensibly Brazilian practice cannot distinguish between erstwhile imported impositions, and those evolved. In tropical geology, deeply weathered etc. natural sands-gravels mostly scarce, crushed-rock fines require grainsize adjustments with washing etc. and turn out very expensive, everything explaining record-narrow filters-transitions.

Three basic construction techniques used alternatively, much influenced by logistics of materials, construction, weather (shrinkage-soaking) etc. Principal discussions and criticisms centered on intuitive contamination, well confirmed not to exist: core CH/MH/SC chunks do not mix dispersed within contiguous sand. Visually cleancut construction uses shallow intermittent trenching for refilling with sand: vertical or inclined equally facile, without/with displacement at each step, superposition for draining continuity easily guaranteed. Alternate expedient advances with sand-fillet trapezoids, followed by spreading-compacting adjacent clay lifts by pad-type rollers. Third expedient, especially in multiple sideward transitions, uses deposition in subdivided metal box mounted on skids, rear steel face windows automatically cutting to desired heights.

Inspections as-constructed appear perfect, but unplastified disintegrated interface of sand-silt-clayey material may be undergoing minute operational progressive erosion-displacement, hitherto altogether uninvestigated. Principal modern transition problem is of great hang-up of differential settlements, raising questions on shear accomodations, possibly impairing continuities of contiguous "columns".

5.3.5. Mid-fill transverse contact slope, phase construction

Restrictions on slopes steeper than about 1 on 3 were initially imported, often still mentioned.

Locally never observed problem: in fact, analogy from much steeper abutment contacts raised question on illogical restriction. Argument regarding different loading-settlements on natural abutments vs. recent-fill abutments also illogical, because of wide variations, most often natural abutments being more compressible. Final attenuation/discarding of concern given by narrow-core earth-rock dams, mostly much steeper rocky abutments: also phase construction on-river separation has been successful with core slopes on US-DS transverse section accompanying (contained by) adjacent US and DS compacted rockfill shells, approx. 1:1.3-1.5.

5.3.6. US-DS galleries

Starting imported practice employed reinforced concrete collars along external interface, employing "creepratio" calculations. Generally recommended independent steel conduit inside gallery, or steel lining: obvious preferences for US gates-valves.

Local and international practice early abolished the collars along impervious stretch because of practical compaction objections, and intuitions on stress redistributions etc. unable to force preferential zig-zag flow. Self-same creep ratio criteria, jointly with flownet reasonings, rapidly indicated much better solution by using filter-drainage wrapping around DS stretch of galleries.

External faces of galleries obviously must employ slight "positive slopes" (forcing compression with settl.) and avoid proportionally high "step", especially under lower dam sections (e.g. < 25 m), principal cases that gave problems lacking compressive settls.

Steel linings mostly dispensed. Hair cracking of concrete accepted as frequent, recognized as presenting no problem, not solvable much more cheaply.

5.3.7. Gravity section transitions

In this item the local evolutions have been so great, theoretical-practical-economic, that summarizing present successful practice should suffice (cf. de Mello 1977). Gravity section slopes have to be positive (approx. 10 on 1), definitely with no dents or slope changes, recognizing relative settls. of fill against face. Early creep-ratio intuitions (absolutely inapplicable, variable from top to bottom ref. contact pressure quality), and favouring of lengthening impervious contact by US face wrap-around, consciously abandoned, strong technical-economic reasons. US face contact tends to open, because of big-reservoir "pendular movements" of dam (cantilevered on reservoir rock bottom), and because of slope obliquities of stress creep and hydraulic cracking etc., effects being increased at more vulnerable higher elevs.

Head-on core contact (special "plastic" materials and compaction specs) against face not oblique to US-DS constitutes present preference, apparently well supported. DS edge and face lined with filter-transitions. Obvious preference (volumes-costs) is US/DS Compacted rockfills, slopes longitudinal preferred somewhat steepened favouring time deformations compressing against face.

5.4. Foundation Treatments

In tropical residuals and saprolites the relatively rapid increase of consistency (incompressibility and resistance) with depth, and the relative ease of excavation to lower foundation elevation, leads to the fact that the only treatment required below the chosen support level is for imperviousness, permeabilities being most often much greater as one transitions from chemically weathered (saprolite) to jointed rock horizons, partly weathered to sound. Geomorphological conditions have almost generally led to central river channels on relatively sound jointed rock, layouts employing mostly in-river hydraulic structures with respective grouting and drainage features.

5.4.1. Grouting of jointed rock

The problem of water-loss testing of jointed foundations, and subsequent cement grouting programs, was tackled with suspicious curiosity since 1955 because it seemed to be the major factor, of crucial responsibility and big highly variable first cost in dam design-construction, left as unquestionable, unquantifiable mystery. In the Iberian peninsula where major arch dams had been rising fast since c. 1947 it was discovered that neither Owner, Designer, nor Inspection as much as inquired into the subject: left entirely to specialized grouting contractors; in some ways the exponentially greater responsibility was fended by profiting of the grout-drainage gallery; presently many cases of regrouting etc. under way after c. 30 yrs. of monitored operation. The Lugeon Test, c. 1932 (5 m test stretches averaging more, 10 bars applied gage pressure) ruled alone, with a criterion of high-pressure grouting all rock of water-loss values higher than 1 Lugeon (1/min x m for 10 atm): intuitively conceded 3 Lugeon criterion for earth dam foundations, presumably based on lower geometric (i.e. average) gradients, although real geometric gradients closer to about 25:1 for concrete gallery case of grout-to-drain "curtains" in comparison with the then narrowest earth-rock core bases. Perforation imposed rotary drilling, intuitively banning cheaper percussion drilling under presumption of chippings clogging cracks.

Purposely entering into grouting (for profit, and for experience) proved that percussion drilling quite acceptable (presently agreed worldwide) and that water loss tests and grouting amenable to theoretical-statistical analysis-prediction. First results summarily proved (de Mello and Cruz, 1959) that there are statistical correlations between averages of water losses and grout takes (cf. Figure 12), the grout takes becoming quantitatively all the more similar to the water losses with increasing crack widths (greater coefficients of water loss, 1/min x m x atm, at judiciously chosen pressures, preferred to direct or proportionally adjusted Lugeon values): important interpretation extracted was that Lugeon limits, besides being unfavorably adulterated by frequent hydraulic fracturing, seemed to be of groutability of rock, not at all logically related with any criterion of need or not of grouting, for technical or economic reasons. The purpose was merely to prove that no mystery hindered rough estimations of grout takes in routines of that time, thus opening doors to statistically quantifying investigations.

Following the thread, Sinclair's PhD Thesis at Urbana, Illinois, 1972 (in extreme concision) proposed logical indices based on measuring differences between water and grouting tests for distinguishing between groutable and non-groutable cracks. The clearly fertile statistical analyses unfortunately failed to advance in professional practice, because of minor pragmatic hindrances: difficulties of double testing etc. in early investigations in remote locations etc.,

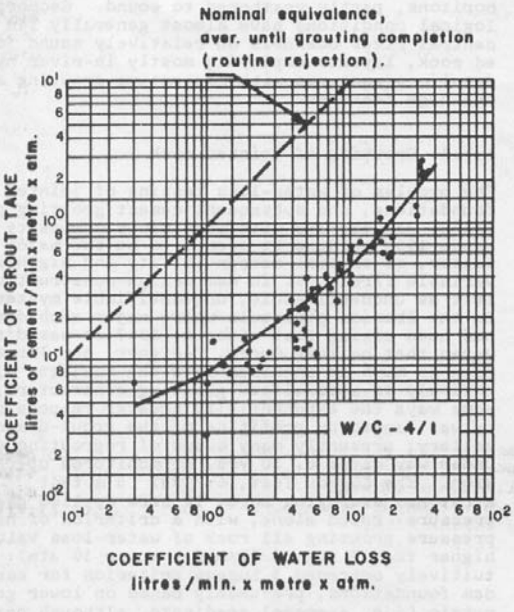
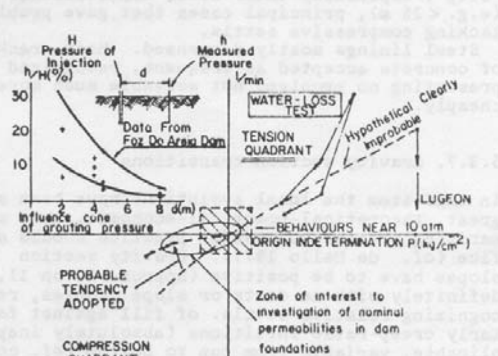


Figure 12

greatly varied groutabilities achieved by finer cements and special mixes etc., and even some purposeful controlled hydraulic fracturing for special grouting cases, etc. Again, the criterion sought was of groutability, not of need for grouting, the really important design decision. Botelho (Brazil 1966) oriented to idealized analyses of various crack widths, published earliest indications of comparative design benefits of grouting vs. drainage, the basic issue of c. 1958-68.

Average flow "reduction ratios" of about 6-10 achieved by "final" split-spaced grout programs, easily explained, despite the fact that individual wider cracks well grouted cause ≈100 % head loss along the specific crack, as demonstrated (for instance) in Itaipu Dam (c. 1982) basalt foundation. The reasoning of truncation of rock foundation's original frequency distribution of crack widths, was summarized by de Mello 1977, in submitting concept of grouted zone as being a more homogenized "grout-buffer" zone in the discontinuous medium, diametrically opposed to the concept of an infinitely impervious, infinitesimal width curtain discontinuity inserted in a continuum (e.g. Casagrande 1961 etc.).

Criteria for pressures on water-loss tests early concluded to have no relation to reservoir head, the test pressure submitting surrounding rock to tensions, while reservoir head mostly not; and, during test, indications desired on three principal points (a) likely in situ permeabilities at very low applied pressures, closest to axis-of-coordinates between tension quadrant and diagonally opposite compressive quadrant (b) practicability of economic grouting (c) critical hydraulic fracturing threshold. Criteria limiting grouting pressures in function of depth, initially imported, and still found wides readly used, were early demonstrated to be absurd, because during period of fluid losses (head losses duly considered) the uplift around given hole is an exponentially reducing "cone" (inverse of well drawdown), and also uplift (analogous to plate anchor pullout problem) is resisted not merely by weight but also by shearing along boundary. Rapidly developed present concepts, including (a) assessing critical hydraulic fracturing pressure via discontinuity of pressure vs. water loss diagrams (b) avoiding, or profitably using, the hydraulic fracturing during grouting, as desired, as a designer's judicious choice (c) emphasizing permissible and desirable grouting pressures during notable fluid takes, and the rejection pressure limit when fluid behaves as Freyssinet jack (d) recognizing that most hydraulic fracturing, except in sedimentary rocks, along subvertical planes, has no relation with overburden weight, and grout-buffer optimization more influenced by reducing total length of perforations while favouring longer fluid travel along wider cracks (cf. Indications of Fig. 13).



HYDRAULIC FRACTURING FOR GROUTING

e.g For Canaliculi In Saprolites

Figure 13

Early imported control criteria against damaging rock, by surface control of heave, rapidly discarded as too remote, imprecise and inconsequent, permitting big attenuations of local hydraulic fracturing, by compressing intervening cracks etc. before transmitting any deformation to surface evidence.

Single-line grouting increasingly favored, many thousands of holes grouted showing invariable exponentially diminishing returns in successive pinching-in and/or rows, as dictated by search for too low acceptance criteria such as 1 or 3 Lugeons. Incidentally, recognizing 1-3 Lugeon crack widths as too fine, the area grouting fear of piping loss of base of core is clearly very grossly exaggerated, since practically no core particles would penetrate, and if they did, total volume eventually losable from base should be very far from risking piping etc. In area grouting under narrow (preferably inclined) cores, obvious priority is for sealing the approx. 0.1H DS foundation of core. In higher dams, overburden weight well contributes by tightening subhorizontal cracks.

Astounding success was achieved in the grouting of jointed rock under the plinth of the world-record Foz do Areia (160 m CFRD) dam (much published, c. 1978-'85). Reconsidering the historic change from concreted cutoff-trench into rock, to the upstream (blanket) plinth (c. 1958) for CFRD projects, it is postulated that a forthcoming evolution of technical interest will turn the plinth apron inwards, obviously incorporating the adjustments necessary, technical, constructive, and logistical. The then overlying select rockfill and filter loading will be a starting benefit, and, above all, the reservoir loading on the concrete face will contribute much, as rapidly as seepage pressures develop within rock. In sedimentary rocks, more problematic, the width of internal aproh can be increased with great benefits.

Many kilometers of 25-60 m high cofferdams thrown on unprepared rock river bottoms, and very revealing respective pumped flows in dewatered "area wells", clearly indicate that the permanent dams' grouting requirements have been much exaggerated.

5.4.2. Impervious blanketing

Conventional US impervious blankets caused great frustrations. Firstly impose guarded consideration because of frequent high permeability anisotropies, partly within stratum, principally by layering (e.g. "pebble-marker", etc.). The early low dam blankets employed gave problems, cracking etc. initially associated with drying shrinkage, later tended by cumbersome construction logistics etc. Subsequent analyses especially on abutments, revealed serious problem of transient water pressure differential above and below blanket, in unsaturated subsoil retarding flownet development to the permanent condition exclusively treated in available publications. Concept of US blanket at zero load suddenly receiving reservoir loading at inoportune first filling of utmost consequence was discussed by de Mello 1977; aggravated conditions concentrate with rapid filling, slow development of flownet required to compress intersticial air, and added compressibility and collapse behaviors, all contributing to subsidence, cracking, sinkholes, etc.

Concept and generalized application of internal impervious blanket presently presumed undisputable, including (cf. de Mello 1977) advantages of (a) possibility of separating flow across dam superstructure and foundation (b) decreasing foundation flow and (c) important, decreasing expensive subhorizontal drainage blanket of avoiding foundation seepage uplift under DS shell zone. Obvious that absolutely no "core-contact" preparation needed under internal impervious blanket, except avoiding sandwiched layer worse than underlying and overlying ones.

5.4.3. Cutoffs by diaphragm walls, or trenching, or saprolite and canaliculi grouting

Cases of diaphragm wall have multiplied greatly, and need no mention except regarding special conditions. Theoretical analyses of idealized influence of "window" in homogeneous medium have absurdly exaggerated the need for bottom socke ing into impervious bedrock: in practice, the benefit from forcing flowlines down-and-up gene rally provides significant benefit, and benefi cost-risk ratio of difficult rock-socketing invites serious reconsideration, including construction difficulties and delays that even aff defects/reliability of upper diaphragm, and including differential settlement problems for ne gative friction on wall.

So-called "plastic diaphragm" using bentonit cement slurries with special rheologically eff tive admixtures, needs important questioning, since most often, after setting, the stress-strain-strength behavior is far from plastic, ther highly rigid and strain-softening, increa cement contents increasing peak strengths and steepness of strain-softening. Appears that m often simple concrete diaphragm is to be prefer including for non-erodibility if any cracking suffered.

Pioneering applications of jet-grouting and jumbo-grouting (soil-cement) techniques have b used both for impervious diaphragms, and for s bilization-cells in proposedly obviating seism liquefaction of loose sandy subsoils under exi ing embankments.

Cutoff trenches of various depths were used since early days, intercepting principally the upper colluvial or elluvial "porous" horizons. Intermediate imported trend, c. 1958-'70, insi ed on "positive cutoff" down to sound (groutab rock, but respective need and benefit/cost rat comprehensibly set it aside, principally becau of mistake of seeking top of rock as presumed more impervious. Partial cutoffs academically criticised, but permeability gradient of reali subsoil etc. reestablished them as useful.

Significant peculiar foundation permeability problem in tropical horizons found to consist canaliculi, of diameters between mms. and abou 5 cms, up to depths of score of meters, and generating networks of tubular flows across distances of score of meters. Considerable natur science research established them to be mostly due to termites and jumbo earthworms, generall perforating subvertical in search of deep grou water (geologic or recent): the "minhoeuçu" earthworms proved able to perforate-digest-eje soils compacted to more than 100 % PC contain coarse-gravel size laterite concretions. Proba bility of subhorizontal US-DS canaliculi not c sidered great, neither that of live survival-seeking lower organisms occupying superposing interconnecting spaces: however, prudence in the first two long dike sections of Amazon-bas projects required guarantees against extreme-v lue-statistics hypotheses of piping failures. Flow-effect testing on samples with canaliculi showed that there seem to be strengthened lini left by the organisms, since even with high no nal gradients (e.g. 50 etc.) there was absolut no erosion of canaliculi walls over periods of weeks.

In the Balbina Dam (c. 1982-'84) project, engineering solutions were required while canali culi were still under preliminary research efforts. For the purpose of sealing them it was reasoned that the only way for travelling grou slurries to find and fill tubular features, wo be to have them travel along planes, that coul be crossed by the canaliculi. Thus it was dec ed to employ a treatment of the said saprolite by a battery of holes fitted with tube-a-manchettes, and employing controlled hydraulic fracturing, for controlled slurry-injection.

In broad terms the treated stretch behaved acceptably, but the treatment (under special research and force-account conditions etc.) resulted being considered expensive. Technically, one difficult problem depended on the starting pressure required to crack the soil-cement sleeve, greatly varying with mix, time (logistics) etc. Further, a curiosity was that successive grouting stages (using different colours) frequently occupied the selfsame earlier crack (suggesting some doubt on the publicised compaction grouting). In my opinion (Consultant generator of the idea) a technical misconception was the use of fixed volumes of grouting per manchette, as is current in permeation of homogeneous sands by assuming an advancing front (e.g. Serre Ponçon, France, c. 1953, etc.): conceptually, travel along cracked planes should be limited by evidences of distances travelled and not by volumes of takes. The concept of this treatment may retain interest for cases below groundwater, but will have to be improved, adjusted and cheapened.

5.4.4. Drainage features. Relief holes. DS rockfill toe. DS drainage trench, and relief wells

DS drainage always well recognized as the single most important feature for guarantee against catastrophic DS failure. Yet early designs conceptually failed in every respect, both regarding "design" dimensioning of those features, or applying direct implementation as proved necessary (analogous to progressive grouting pinching-in), or considering their monitoring with time with recognition of loss of function, or providing facility for any maintenance if function progressively impaired (as in gravity dam galleries). Note the all-important systematic periodic testing of responses of relief holes in galleries as adjacent ones are closed (measuring pressures) or opened (measuring flows), and, correspondingly, drilling additional holes as indicated.

Early designs automatically employed geometric distribution, unreasoned, subjective (e.g. in rock, NX holes every 5 m, in some saprolites, 40 cm relief wells, with filters, every 20 or 50 m, etc.) and directly under the embankment's intercepting chimney filter-drain. No water testing, no on-site revision, no monitoring, no access through high overlying embankments. Such inconceivably unsupported practices still widely prevalent, based on visual retransmission of pseudo-precedent. Presently, accepting inexorable subhorizontal drainage blanket on foundation support of DS shell, emphasize that (except in special cases of very anisotropic problematic subhorizontal shearing rock, for which case gallery and filter wells along upstream edge of drainage blanket should be considered) there is no advantage or need of relief except near DS toe.

Interesting monotonously repeated design feature, technically absurd and construction-economically cumbersome-costly, that has been visually transmitted to most design cross-sections even along soil abutments, is the DS rockfill toe. Published designs, books etc. limit themselves to max. ht. dam section on rock river bottom, where the DS rockfill appears mostly because of cofferdamming. On abutments the toe rockfill has no possible function except anathema of permitting dangerous long-term hiding of eroded fines inside rockfill interstices, so that final eventual accelerated piping comes as incurable belated surprise.

Vertical drainage relief is often desirable near DS toe of dam, despite drainage blanket, to control anisotropic transfer of seepages to distant uncontrollable DS boils etc. Because all seepage problems are always preferential and erratically localized (astounding digression from flownet theory idealizations, directly valid only, by averages, for seepage stresses on mass deformations and unstabilizations), the technically best and cheapest solution is to employ filter-drainage continuous (backhoe) trench. At depths of 6-10 m (easy), such trench controls uplifts approx. double the depth, generally ample. If underlying pervious layer feared, judiciously drain it by relief wells from bottom of trench. All wells should come up to accessible top with casing, for monitoring and occasional maintenances.

In light of stated incomprehensibly unapplied test-analysis criteria, emphasize that drainage features must be based on water tests, crack widths and spacings, permeability tests etc., for the most cogent design-construction reasons, and not merely drawn on unsupported geometric appearances from other projects.

5.5. Misbehaviors and failures. Instrumentation and monitoring

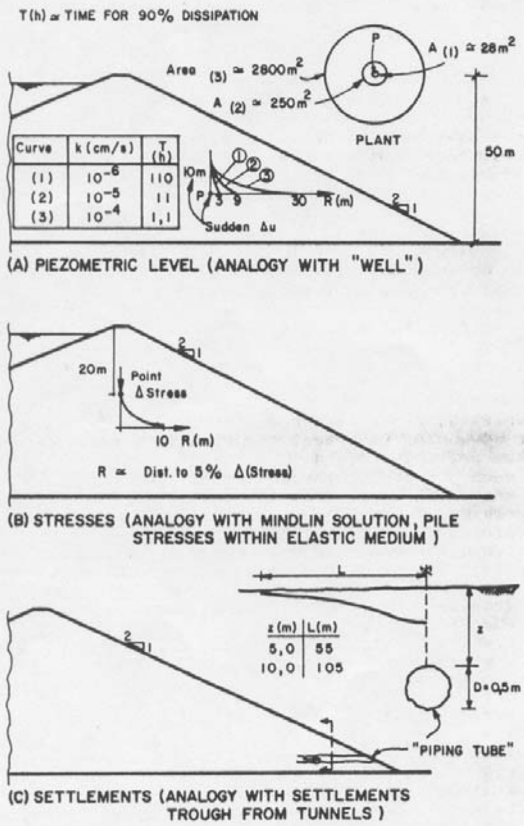
There have been a few important failures: however, their analyses classify them as clearcut textbook failures due either to (a) yet insufficiently divulgated theoretical revisions (b) the responsible parties having respected early idealized publications, manuals, etc., of the early 1950's, as scientific truths, unquestionable or unadjustable.

This author's portentous admonition is that no presently marketed instrumentation, and respectively planned installations, can be relied upon for alerting on impending preferential, localized, failure scenarios. Instrumentation presently in use can only be planned, installed, and later analysed with regard to average theorizable scenarios, and the thereby corresponding trends for advancing (average) theorizations and acceptance criteria (cf. de Mello 1977, 1983, etc.). In such a respect, the entire cultural setting of the civil works from the developing countries supplying the data, generally conditioned by the developed world's professionals supplying the design decisions, has inexorably trended towards restraining evolution, under the deeply inbred need for repeating the proven conditions, even if grossly conservative (impossible to prove if impeded from so doing).

Unless and until civil works of such responsibility as dams are consciously able to plan gradually evolving designs, adequately instrumented, in order to test the frontiers of impunity via "observed, controlled and controllable-revertible degress of misbehavior", it must be emphatically repeated that most instrumentation hitherto has been an incalculable waste, of zero benefit/cost ratio. Instrument and installation costs are an iota compared with the continually accruing costs of observing, plotting, etc., with no consequence: we necessarily exclude mention of the eventual occasional catastrophic risk-cost of presuming safety to be guaranteed while some hidden deleterious phenomenon may be growing.

With respect to alarm-signalling, one problem will herein be considered as resolvable (although at hitherto unestimated extreme cost): that is the continuous-recording and continuous-attention, so that prospective eventual incipient failure would be instantaneously observed-reflected. The problem of speed of action, for counteraction, will be set aside herein, so as to minimize frustration. There are still at least two questions of attenuation-delay of transmissions, of point observations, to be picked-up, and to represent, the behaviors of the surrounding mass: one is the simple problem of transmissions within an elastic (instantaneous) medium; the other is the time-attenuation based on the material's rheology (rather more difficult and variable).

Figure 14 provides indications on the single, simple, instantaneous-elastic problem, by using the applicable analogies with available analytical solutions. Suppose that at some point the pore pressure "suddenly" increases by a significant value: by using such analogies as



IDEALIZED RANGES OF RESPONSE OF INSTRUMENTATION OBSERVATIONS

Figure 14

the solutions for wells, we can readily see (Fig. 14a) that the alarm-signal would not be picked-up in any more than minor proportion by any piezometers beyond a certain radius. Similarly by using the classical solutions of stress applications within an elastic medium (e.g. Mindlin etc.) we reach the same conclusion (cf. Fig. 14b) that no significant proportion of the applied stress would be recorded beyond any modest distance. Finally by using the classical solutions of the opening of a cavity within an elastic medium (e.g. Carrillo's stress centers 1942, or all tunnelling data on settlement troughs etc.) we reach the same sobering conclusion that any piping or karst cavity developing at greater depths would be difficult to perceive on the basis of surface settlement measurements. Many other more sophisticated available solutions can be used to lead to identical conclusions: point measurements, of whatever type, cannot be used for assurance against preferential failure scenarios. And, presumably, in dams that avoid textbook failure conditions, the real intent and purpose of instrumentation is to document with respect to theorizable scenarios of "degrees of misbehaviors".

Recent developments are suggesting mostly new lines of instrumentation and monitoring. Some such are: (a) profilemetering and increased interest in internal lateral deformations: (b) measurement of suctions (with every attention at minimizing the greatly conditioning interferences of soil-instrument installation interface conditions): (c) investigating and confirming the compaction precompressions of angular rockfills: (d) investigating the perspective of monitoring with sensors of special design (for erasing background "noise" and precision, the possible microacoustic generations as indicative of incipient failure.

Great advances have been made in embankment dam design and construction, but because of misconceptions regarding the true intents, purposes, and fruits, from purposefully developed- installed-calibrated-interpreted instrumentation, there are yet great strides ahead for optimization of cross-sections that, strangely, continue geometrically very similar since 40 years ago.

6. CONCRETE FACE ROCKFILL DAMS

6.1. Introduction

The progress in the design and construction of Concrete Face Rockfill dams has been one of the most notorious in Dam Engineering in the last twenty five years.

The major improvements are based mainly in two facts:

a) The use of efficient vibratory rollers in the compaction of relatively small lifts of rockfill. This practice, started in the 60's, avoids the uncontrolled large settlements of dumped rockfills.

b) The adoption of a semi-pervious finer material in the zone immediately under the face slab to provide a secondary control of leakage through slab joints and adequate support for the face slab (Sherard, 1985).

Major improvements are also the currently adopted practice in the design of foundation cutoff and concrete face slab and joints, abandoning the previous design of highly articulated, checkerboard pattern of slabs and compressible joints (Leps et al., 1985).