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A case history of a major construction period dam failure

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INTRODUCTION

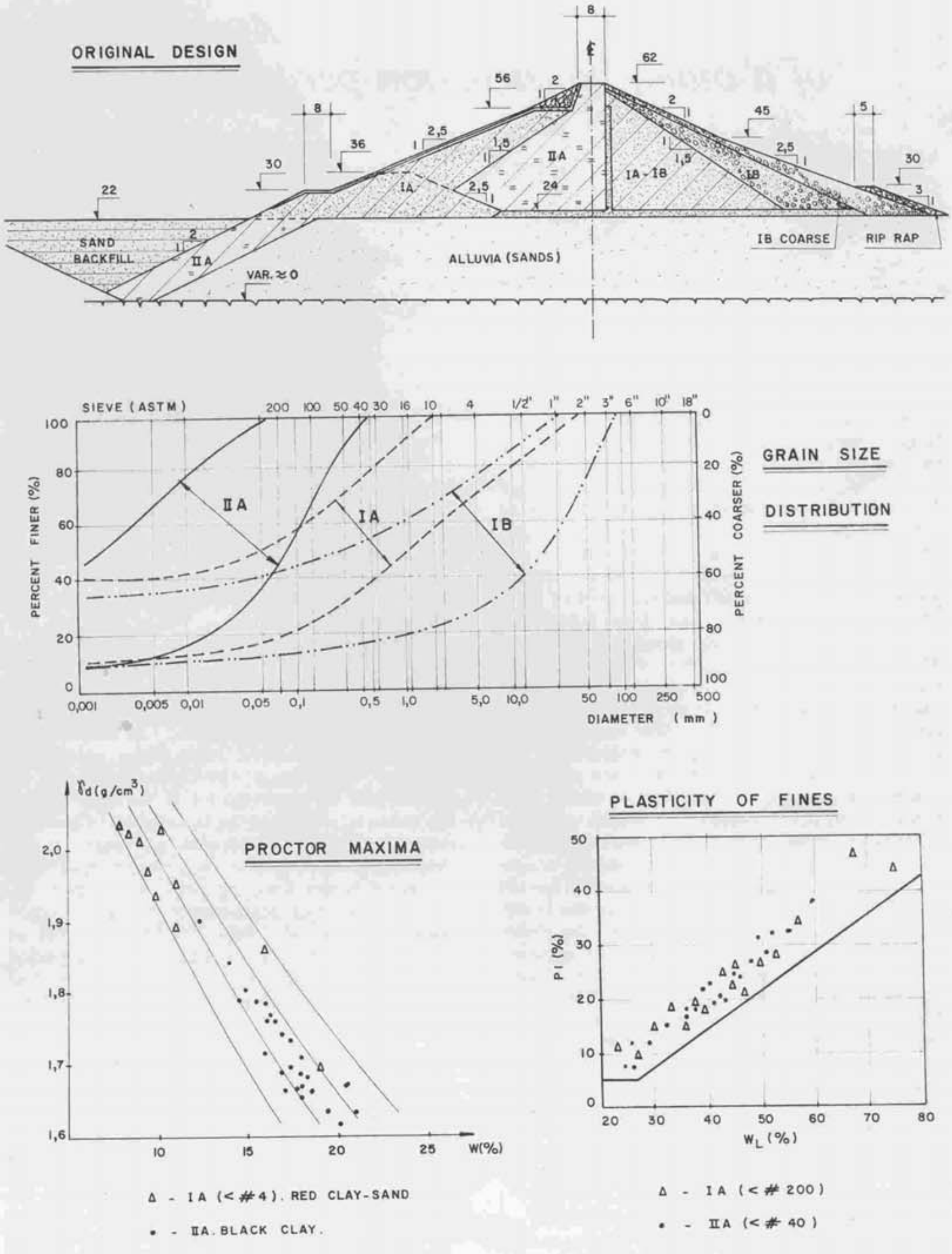
In the recent past there have been some very disconcerting failures that, upon analysis, tend to indicate the onset of an unfortunate cycle of misplaced application of geotechnical knowledge. Many a case may be recalled, and in most of them the post-failure analyses unearth a consistent undertone that would almost shame one into apologies for a possible injustice of a-posteriori judgements. Truly, though, the sequence of identifiable errors is bewildering, even when surrounding an old and well-recognized problem. The impression arises that the spread of geotechnical analysis-synthesis has reached circles quite insensitive to the fundamental behaviors and the conventional simplifications. Sow the wind and reap the whirlwind. Our great mentors of the early days of soil engineering faced the humbling complexities of the unquantified problems, and made an effort to achieve conventional solutions, that they well recognized as conventional, idealized, and simplified; thus, when applying a simplification they carried with them the full benefit of the wisdom of those that start from the bewilderment of reality and painfully reach the ability to distill it to the essences of simplicity required to solve the problem. A new generation of geotechnicians has been taught the simplified solutions, oft without sufficient emphasis on hypothesis and recognition of closed-cycle conventional practices, and so the rational simplicity of rationalizations has suppressed all humility towards Nature. Also, what is insufficiently understood has been totemized. Then, time and again, suddenly one is shocked into the realization of how dearly Society will pay for old problems, of the classical 1940's, erroneously handled.

One such case is herein summarized with an attempt to pick out the fallacies that accompanied an intended high-level cognizance and application of soil mechanics and earth dam engineering.

ORIGINAL DESIGN AND SPECIFICATIONS: START OF CONSTRUCTION

The principal problem faced in the design of the dam shown in figure 1 was recognized to be the need of a positive cutoff across the sandy alluvia. Two types of borrow pits were identified and investigated in conventional laboratory tests: one was a dark-gray to black flood-plain silty-clay; the other was an unsaturated terrace red clayey-sand with gravel. It was recognized and conventional triaxial tests duly confirmed, that the blackish silty-clay had poor shear strength parameters, and thus its use was confined, both in the impervious blanket backfilling of the cutoff trench and in a central core. For more appropriate and guaranteed construction scheduling between flood seasons the cutoff trench was situated near the upstream toe of the dam, so that its construction could be developed in parallel with some of the embankment placement; it was reasonably forecast that such construction, with its groundwater lowering and rock-contact surface and grouting treatments at the bottom could face problems and delays. The upstream shell of the dam was designed to employ the compacted clayey sands and clayey-sand-gravels that had been estimated, and proven by conventional [1]

[1] All over the world one encounters the problem of misplaced concepts regarding standardization of laboratory tests. Standardization is necessary for *consistent communication*, and early standardizations had such a merit despite the ponderous limitations of dictates emitted when knowledge was barely beginning to be gleaned under crude laboratory equipment and modest comprehension of phenomena. One should emphatically guard against the illusion (a world-wide bane) that standardized tests purport to reflect "*real properties and behavior*", rather than merely a covenant of *nominal properties and behaviors* for organized communication. In both the materials in question laboratory routines impose very significant adulterations of field conditions. In the gravelly soils it is the need



Basic data, original design and index lab. tests on materials
FIG. 1.

laboratory tests, to provide a highly resistant and amply impervious material. Thus, an upstream partial embankment of the red gravelly material would be raised on top of the impervious element of the trench, to serve as upstream cofferdam against floods : this impervious shell would also tie the cutoff to the black clay impervious core. The remaining features of the dam are routine and need no comment.

The borings in the alluvial sands revealed some fine gravel sizes, principally towards the bottom. There are geomorphological reasonings favouring acceptance of erratic gravel lenses, and, principally, basal gravels. It must be recalled also that in many a wash-boring there tends to be an accumulation of coarser sizes at the bottom of the boring because of segregation due to wash-water circulating velocities. Thus the data from borings can be pessimistic, leading to desirable conservatism. The fact is that visual inspection of the excavation faces of the trench and of the unwatered alluvial plain constitute the incomparably important data : there were found to be no gravel lenses but only interspersed fine gravel sizes in a deposit of perfect filter sand.

The settlements of the dam on the sand foundation were computed to range between 20 and 40 cm in a classic dishshape transversally. Apparently because of such differential settlements (presumed significant ([2])), and because of a now predominant notion that in all materials and compaction conditions a more plastic behavior (and less " cracking ") is achieved by wetter compaction, the bid documents suggested compaction in the range of optimum $\pm$ 1.5 %. The suggested specifications were to be revised on the basis of field compaction behavior and test data.

The bid documents included some recognizedly questionable (fig. 2) geotechnical analyses, but, to an experienced earth dam engineer, visual appreciation of the cross section and analyses should suffice to indicate overal acceptability of the plans, and easy design-as-you-go adjustment. Construction period analyses were stated to have used $\bar{B}$ coefficients (a questionable " constant " in itself) of 5 % and 10 % (presumably a weighted average for both upstream materials, so very distinct, an undesirably crude assumption and extremely low values for the black clay). Rapid drawdrown RDD (upstream) and full reservoir (RES) (downstream) slope stabilities employed flownets and presumed effective stress parameters. The RES flownet seems far too pessimistically hypothesized : in part it is recalled that the assumption of impervious boundary at the top of alluvial riverbed is indeed applicable at two transverse planes, at the bottom of the abutments where the dam rests on sound impervious bedrock ; but the further assumption of an ideally clogged chimney filter-drain, so that the flownet would exit in the narrow downstream gravelly shell could hardly find justification. The RDD flownet is quite unexplainable. Figure 2 summarizes the flownets and nominal stability analyses, run by computer. It was decided that for the submerged slices it would be more practical to adopt $r_u = u/\gamma z = 0$, together with using submerged unit weights : possibly the tabulated use of zero unit weight of water was associated with such practice. Finally, for so-called *long term analyses* a frequently quoted obligation (however absurd) to adopt $c' = 0$ was used ; consequently, the routine computer analyses inevitably singled out as least stable the critical circles tangent to the slopes, for slide volumes $dV \rightarrow 0$. The hypothesis and analysis constitute an extreme lower bound, obtainable at sight by $FS = \tan\phi'/\tan i$.

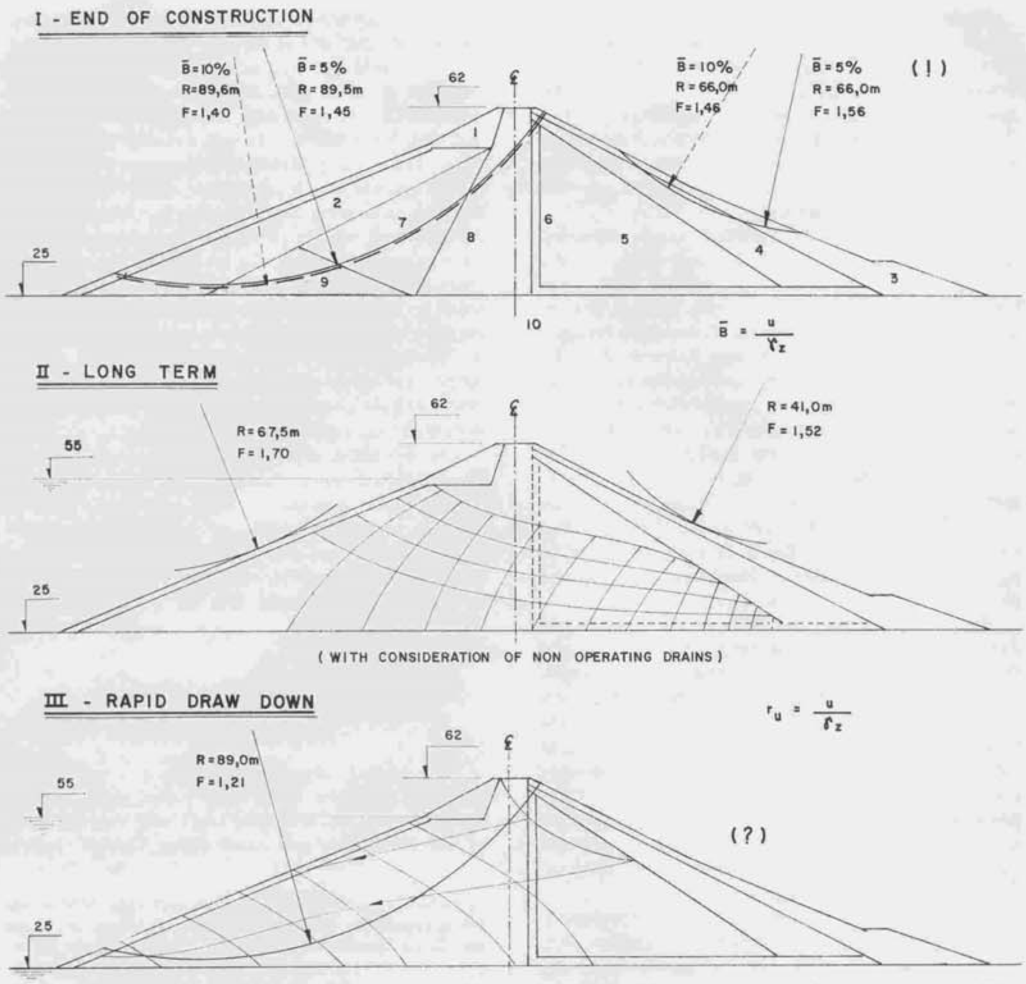
On the unconservative side one must note the incorporation of a most current misconception, spread under routine teaching and texts. That is, the use of effective stress (c', ϕ') coupled exclusively with flownet porepressures, as if under incremental stress-strain conditions that could lead to sliding there would not be a Δu associated with compressibility.

HUMAN DISCONTINUITIES IN ENGINEERING ORGANIZATION AND CONDUCT

The owner organization has a considerable backlog of dams built and other dams under construction : its technical staff includes teachers of soil mechanics and earth dams. Design reports

to exclude coarser sizes even when they constitute an absolute continuum of grainsizes, and, in such exclusion, the significant hysteresis that can be imposed on the clayey fines, and on some crushable coarse fractions, if the sieving is done after drying and pulverizing. In clayey material, even if all testing guard against such hysteresis, the fundamental discrepancy is that the tests in question remould and plastify the soil and later compact it confined in small rigid moulds, thereby disregarding, on both counts, the well-recognized and emphasized importance of *micro-structure* in geotechnical behaviour.

([2]) The author has repeatedly mentioned (cf. fig. 3, Rankine Lecture, Geotechnique, [de Mello, 1977]) that, most surprinsingly, dish-shaped settlements of a couple of meters have been accepted unquestioned when they affect the upper part of a dam supported on its own lower part, while similar settlements of a few decimeters have been conjured to cause concern when they are recognized as due to a *foundation*. Moreover, as will be discussed further, no distinction is made between longitudinal and transverse differential settlements and cracks, or between open tension cracking and tight shear plane displacement. Finally, the fallacious associations between plastic behavior and plasticity indices and wet compaction will also be recalled [de Mello, 1980] because of direct bearing on the case history.



			I ($\bar{Q}$)			II ($\bar{R}$SAT)(1)			III ($\bar{R}$SAT)(1)		
MATERIAL			γ_n (t/m³)	c' (t/m²)	ø' (°)	γ_{sat} (t/m³)	c' (t/m²)	ø' (°)	γ_n (t/m³)	c' (t/m²)	ø' (°)
1	ROCKFILL	(2)	2,0	0,0	38	2,0	0,0	35	2,0	0,0	35
2,9	CLAYEY SAND, FINE AND COARSE		2,0	3,0	30	2,1	2,0	32	2,0	2,0	32
3	PEBBLE WITH GRAVEL	(2)	2,0	0,0	35	2,1	0,0	35	2,1	0,0	35
4	SANDY GRAVEL	(2)	2,0	0,0	35	2,1	0,0	35	2,1	0,0	35
5	SILTY SAND, FINE, LITTLE CLAYEY	(3)	1,9	3,0	28	2,0	1,0	32	2,0	1,0	32
6	SAND	(2)	1,9	0,0	35	2,0	0,0	35	2,0(4)	0,0	35
7,8	SILTY CLAY	(3)	1,9	5,0	15	2,0	3,0	20	2,0	3,0	20
10	SAND, MEDIUM AND COARSE	(5)(2)	1,9	0,0	27	2,0	0,0	36	2,1	0,0	35

(1) BACK-PRESSURE SATURATED
(2) ESTIMATED, NOT TESTED
(3) NOTE DIFFERENCES (QUESTIONABLE) BETWEEN TWO TESTS OR ESTIMATES(5)
(4) γSAT

Original design, basic strength parameters adopted, flownets and stability analyses

FIG. 2.

and bid documents were accepted, and there was a delay of about 3 years before bidding, and another year before start of construction.

Tenders were also put out for field inspection and design-as-you-go engineering. The designers were among the bidders, but lost. As inspection started questions were raised by the Inspectors, but the Designer understandably declined to comment. A solution for the desirable tie-in of the design and construction phases was sought by engaging the individual consulting services of a geotechnical engineer who had had a key participation in the design but had since left the firm and established himself as an individual consultant and part-time professor. The Inspectors further engaged an international specialist consultant to assist them in contractual obligations " to offer specialized services of design revisions if and when the optimization of the project would suggest or require them ". Administratively there were significant discontinuities in the professional attributions and chain of responsibilities. These and other administrative problems may well be singled out as the most persistent cause of the failures faced.

It is quite astounding, firstly, that Owners do not recognize that when contracting the services of a company (either in series or in parallel), to " optimize design-as-you-go revisions " of the design of a competitor company, they inexorably invite revisions out of jealousy and dispute rather than out of need or zeal; it is in animal and human nature. Engineering design is an art, quite subjective, and not a right-or-wrong objective science. Secondly, when the Owner's technical staff perceives that the " specialists " are dedicated to picking bones, it is tragic that no benefit be sought from the dispute; the old motto " divide et imperat " requires the strongly deciding " imperat " to cull the benefits from the dispute.

One must emphasize that the construction scheduling was necessarily speedy and tight, and to a contractor, fortunately, the rules of the game are " Le roi est mort. Vive le roi ".

SIGNIFICANT DESIGN REVISION SUGGESTED, DOCUMENTED, AND DISCLAIMED

While the execution of the cutoff trench was pushing ahead, the international specialist consultant was firstly engaged by correspondence to revise instrumentation ([3]) plans, and to revise the " flow-net analysis of the as designed section ". The unexplained boundary, conditions of the design flownets, and their further failings, doubtless invited attack. There resulted a special consulting visit by the international specialist, and a significant revision of the design section was suggested essentially along line finally adopted as shown in figure 3.

The reasons for the need to revise the design section were stated to be a cause of " major concern " and are summarized as follows.

Firstly it was questioned why the cutoff trench was not, as is usually the practice, nearer the center of the dam, under the impervious core zone ([4]). Secondly, by visual examination and by three grainsize tests it was concluded that the gravel contents of the gravelly clayey-sand were too high, so that the Zone II material might not be impervious ([5]) to constitute an acceptable connection between the black clay cutoff and core. Thirdly, there were repeated indications of concern regarding differential settlements and cracking and high seepage gradients ([6]) such as to make it good that the core material be a plastic (CL to CH) soil, with placement wet of optimum ([7]).

([3]) One might well reflect on the irony of how often the most subjective and most frequently misplaced and wasteful engineering effort (cf. Rankine Lecture, [de Mello, 1977]) is used as a first breach for design revisions. Thereby one adds to all the original failings of instrumentation designs, the further bane of delayed ordering and belated installation.

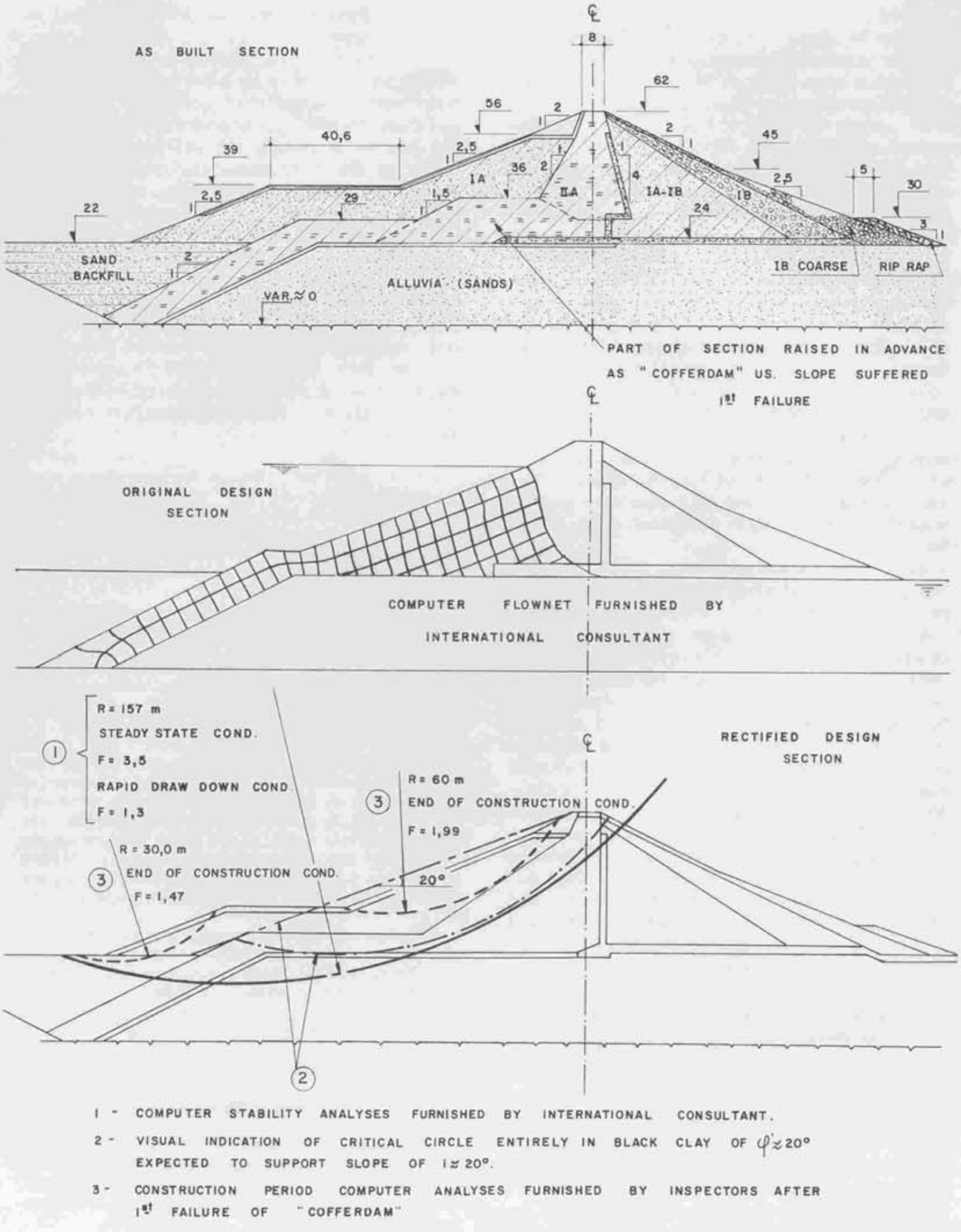
([4]) Although a geometric analysis of most dams would indeed have placed the cutoff as stated, it should be recalled that with regard to the Teton dam failure both the Special Panel's report and the Bureau of Reclamation's self-criticism recognized the cutoff trench's differential settlement problem, arching, and craching, as a significant design error aggravating risk of failure on first filling.

([5]) Visual inferences based on indirect index testing should never substitute direct tests in situ when construction is agoing. Gravelly materials have led to surprises of very low permeabilities principally because the continuity and shape of the grainsize curves (and grains) has not been considered. Again, in the Teton dam failure the permeability tests revealed that what was meant to be a gravel drain furnished results mostly in the " practically impervious " range, (cf. for instance, [Zeller and Zeindler, 1957]).

([6]) Regarding allowable seepage gradients the profession seems to have mostly forgotten that seepage stresses are vectorial, and that direction (compressive vs. tensile) is of prime interest (ref. piping etc.). Moreover, it is generally accepted that even high gradients should not arouse concern if there is a good filter backup. Finally, a gradient of roughly 3 across a blanket or cutoff trench would be considered quite low in comparison with conventional designs.

([7]) The increasingly heavier earthmoving and compaction equipment have made it quite incompatible to associate plasticity indices with plastic deformability of site-compacted materials [de Mello, 1973, 1980, 1981]. CH materials well compacted tend to be " dry " and brittle; when compacted " wet ", intense laminations of overcompaction tend to result.

Regarding specifications of allowable range of compaction moistures, the practice of using members (eg. $-2\,\% < \text{opt} < +1\,\%$) has been the source of very repeated trouble since materials used over the world have ranged in



Rectified section recommended and documentation

FIG. 3.

Thus, two alternate schemes were proposed to rectify the design shortcomings discussed above. However, before either scheme was adopted it was recommended that upstream stability analyses be made for both schemes for steady state and drawdown conditions (four analyses). The questionable analyses and borderline safety factors of the design documents were the reason for the requirement. A buttress of the upstream toe of Zone IA material was recommended to increase the path percolation and to strenghten the toe stability because of the replacement of Zone IA clayey sand-gravel soils with more plastic black clay soils (IIA).

Although the rectified design, to provide a continuous connection of fat plastic clay at the base of the upstream section was recognized to reduce stability, no mention was made, at any time, of construction period stability analyses, either on the proposed section or on ensuing discussions and ratification. The flood-plain borrow area (submerged every year) of dark gray plastic clay (generally CL to CH) and the core placement were inspected and the material and the construction procedures were declared to appear to be good ([8]). It may be remarked as ironical that the powerful modern excavation and hauling equipment on big earthmoving jobs has generated a new problem often serious. More heavily preconsolidated clays (higher dry densities) can be excavated easily, and even indicate a need of watering (cf. fig. 4) to bring the moisture content to Proctor optima; if the soil is close enough to saturation, what the compaction equipment does is to remould, shear and laminate the clay, and not compact it [de Mello, 1981].

The " necessary " backup design computations for the international consultant's recommendations were conducted in the office of a very major foreign consulting company using the soil parameters that he assigned, and acting as engineers for himself. In each case a computer program was used and reference thereto was limited to a program name and supplier company ([9]).

Moreover regarding tne soil parameters used it was pointedly stated that " I adjusted some of the soil parameters for strength based on my experience with some similar soils tested in the past ", and " I used a higher value of cohesion than the designers used in their analyses. As a result " the upstream section of the dam should be stable with either of the two alternate modifications... ".

The computer analyses submitted included a finite element flownet of steady state seepage of the original design section, presumably to justify the concern regarding the alleged high seepage gradients (fig. 3). It included a curious and obviously unnecessary proviso regarding possible interference by silt layers in affecting the drainage into the alluvium (although any such layer would merely constitute an insignificant increment to the clay blanket thickness); this serves to show the effect or a loss of identification and feel for the physics of the problem.

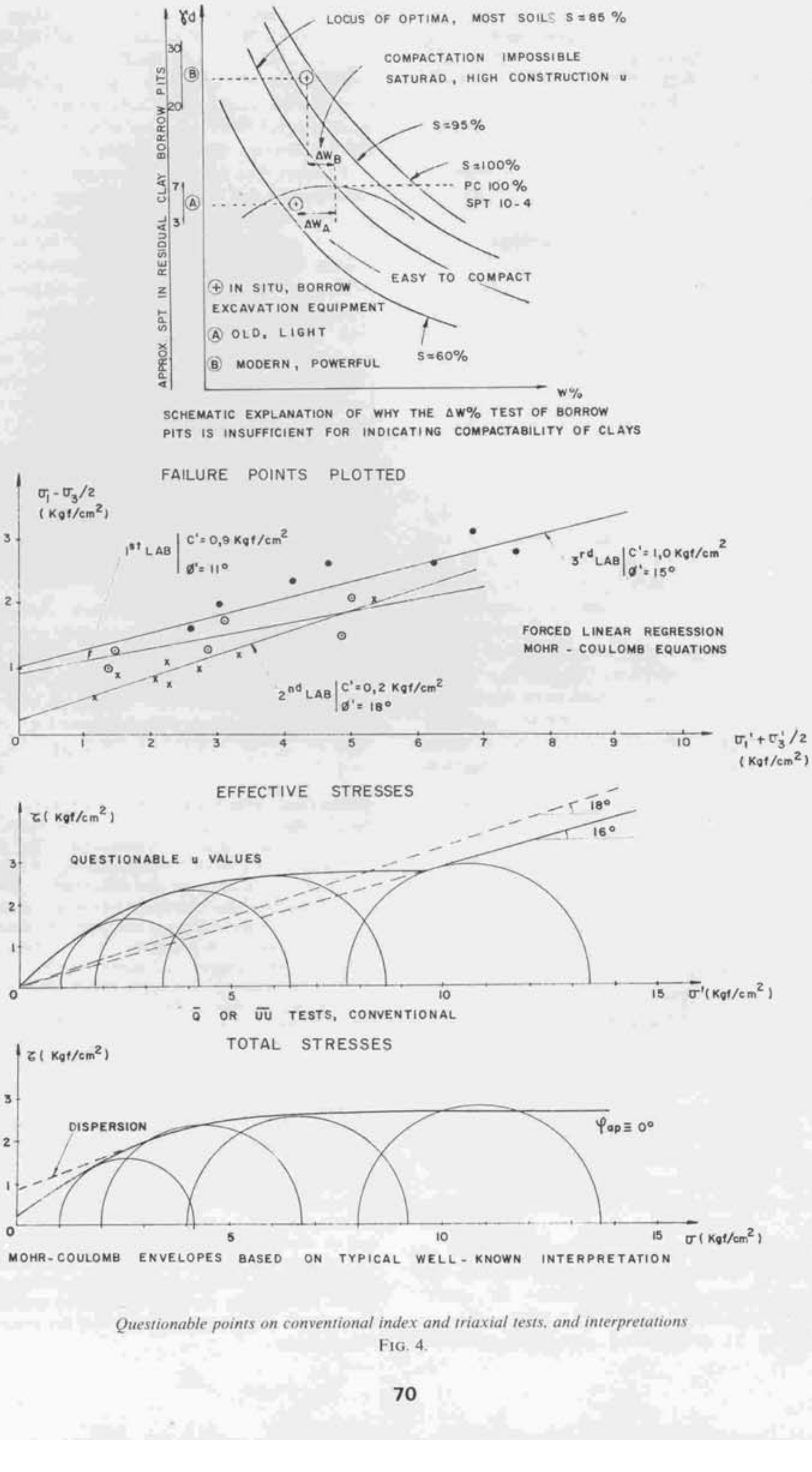
Further, the only other computer analyses submitted included the upstream slope stability cases (fig. 3) confidently considered applicable and sufficient. Regrettably the manner in which such results are furnished do not facilitate independent appraisal. However, some significant criticisms transpire immediately, such as: the cohesion parameter adopted, and the manner in which any possible soil testing was set aside; the oblivion of a construction period analysis, even in comparison with a full reservoir upstream analysis on which the absurdly high FS = 3.5 should serve as a minimum hint; the fact that failure " circles " did not establish themselves with a maximized stretch in the black clay, recognized as the weaker material and the only one definitely non-draining (moreover, the fact that the " critical circle " was given as coinciding for both the conditions).

Finalizing such a definitive consulting intervention backed-up by design computations, the report served the Owner with a truly astounding disclaimer general statement to the effect that the consultants were not " involved in the design of the dam nor were commissioned to review the

optima from about 8 % to 45 %; the erroneous habit-forming communication can be allayed by the use of a fraction of the respective optimum water content, e.g. the reasonable rang being generally (0.93) opt. to (1.05) opt.

([8]) In such a material that frequently occurs close to saturated in situ and wherein drying is only at the surfaces of clods subjected to intense shrinkage cracking, it would have been essential to check the percent saturations in situ since one only compacts air pores. At any rate, it is surprising that mention be made of a first layer moistened to near the liquid limit moisture (very wet) worked into the smooth rock contact and that two additional layers at about optimum +2 to +4 % moisture be placed above this. For any excess construction pore pressure development and stability it is well known that a single weak preferential surface has very often been the culprit.

([9]) Such practice should be promptly and definitely banned. To the lay, the very mention of a computer not only conveys assurance of infallibility but also merely serves to justify highly increased invoicing. To the geotechnician user, the summarized designations (e.g. using the Modified Bishop Method) serve to bury any evaluation of the applicability of the physical model (e.g. rigid-plastic body rotation) but also of the mathematical hypotheses involved. No amount of computer analysis can substitute for a single personal analysis by traditional longhand routines, to furnish the feel for the behavior under investigation.



Questionable points on conventional index and triaxial tests, and interpretations

FIG. 4.

design. Our present work was solely to review the construction and provide any suggestions which, at this date, would provide a better and/or more economical structure. In no way should it be inferred that I have approved the present design of (the) dam ".

RATIFICATION OF THE REVISED DESIGN, FIRST CONSTRUCTION-PERIOD FAILURE AND BACKANALYSES

The local consultant was requested to examine the newly proposed section under indications (questionable) that the red clayey-sands with gravels were becoming scarce. No reference was made, at the time, to the international consultant's reports and analyses, a possible lack of communication. But, for reasons of prospective economy (shorter haul from the black clay borrow) and a concern with differential settlements, a third alternate was proposed " similar to alternate 1 " of the international consultant. Discussions extended for about 6 weeks and finally 4.5 months after the first proposal, a section was adopted by consensus at a meeting. The Inspectors issued the assurance that although further stability analyses would yet be run, construction should proceed directly with the approved section because its stability was assured ([10]).

In anticipation of the rain-and-flood season it was decided to advance the raising of a central section of the core, to serve as cofferdam for part of the work area. Thus, in slightly more than one month the black clay compacted fill was raised from elevation 22 to 36 on a 1 : 1.5 (V : H) upstream slope. And suddenly two slides developed, concomitant, along lengths of about 150 m each, separated by a stretch of tens of meters. A construction-period failure was recognized and the need to revise parameters and analyses was declared.

Thus ensued the most tragically absurd sequence of exchanges of geotechnical debates, until, about one year later, the major construction-period slide took place over a total length of about 600 m, absolutely homogeneous from end to end of the dam about to reach the crest.

Firstly the local consultant ran back-analyses by Taylor's (1948) stability charts, using total stresses ([11]). The c, ϕ values were declared to be much lower than those adopted by the international consultant (who had failed to use the convention c', ϕ'). It should have been recognized that his parameters, which had been used associated with flownet pore-pressures should more reasonably have been considered nominal effective stress parameters. Thus irrespective of the fact that his estimated parameters were untenable, the fact is that there was a regrettable lapse in comparing c_u, ϕ_u vs. c', ϕ'.

Attention concentrated on the immediate reconstruction slope of the black-clay " cofferdam part of the dam ", and it was concluded to require a 1 on 2 slope, (and was even executed as 1 on 2.5 by the Contractor, to gain time pending the outcome of discussions). Full three months later, with the oddest visceral discussions still raging, a revised section of the rectified section was ratified at a meeting called to establish consensus : the direct consequence was to flatten the berm slopes for construction period stability, and the Inspectors (with their international consultants) were charged with the responsability to document the stability, which was asserted to be assured.

It would be tedious to expatiate on the discussions, despite the surprising lessons on levels of misplaced cognizances and preoccupations. In short, they centered on comparative use of total stress vs. effective stress analyses as standardized design criteria ; occasion even arose of having Terzaghi's effective stress principle transmitted by international telex. Discussions further raged with regard to validity of the samples, respectability of laboratory tests, and questionability of inspection tests (both regarding consistent errors and concerning dispersions on the very small-dimension specimens).

In summary, it may be stated that the principal problems affecting the subsequent problems were : lack of top-level technical communication and excessive recriminations on quite secondary issues ; conventional laboratory tests routinely conducted and interpreted by independent " specialist " laboratories, with no sense of adjustment to the problem ; routine adoption of straight-line (c, ϕ) parameters irrespective of stress type and ranges etc. ; routine adoption of circular failure ; lack of sensitivity to the noticeably critical material, the black clay, both regarding intact shear and pore pressures, and further regarding behavior along fissures and laminations.

([10]) It may be instructive to observe (fig. 3c) that the external slope corresponds roughly to 20°, the ϕ' value attributed to the black clay by all involved. A hypothetical " circle " could develop entirely within this clay. Therefore, were it not for the cohesion parameter, even without any pore pressure along the circle the 20° obliquity would be entirely absorbed. Should not visual examination indicate the need to reflect very carefully on c' and u values ?

([11]) It is surprising that on two simultaneous text-book slides involving a total earthmoving volume of about 160000 m^3, the back-analyses should have been limited to such simplified hypotheses as are incorporated in those charts.

Thus, for instance, confusion was such that four months after the construction period failures had occurred in the black clay, the results of construction period analyses, required of the redesigners, were submitted indicating high FS values (roughly 2) cutting across the red clayey-sand-gravel capping ! ! The black clay was once again not analysed for construction period stability. Neither was it reanalysed regarding revised RDD conditions.

One decisive outcome was the preference to abolish wet-of-optimum compaction. The specifications were changed to the range opt. -1.5 % to opt. $+0.5$ %. None of the intervening geotechnicians took special note of the fact that over the entire horizontal extent of the " internal blanket " the lower part had been compacted under the wetter conditions, and could expectedly establish a preferential sliding plane.

MAJOR CONSTRUCTION-PERIOD SLIDE

Almost exactly one year after the above construction-period sliding, the entire dam developed a major slide as the fill reached 5 meters below the final crest. Eye witness descriptions of the failure, which developed in about 30 minutes, vividly depict the sequence, starting with tension cracking at the top (more brittle behaviour), and followed by a rotational slip in the higher reach and a massive pushing outwards of the flatter lower section and berm along a distance of about 25 m. The photos 1 through 5 are self-explanatory ; they were taken 6 days after the sliding, and so the shiny wet slickensided surfaces exposed to the wind and hot sun had mostly dried.

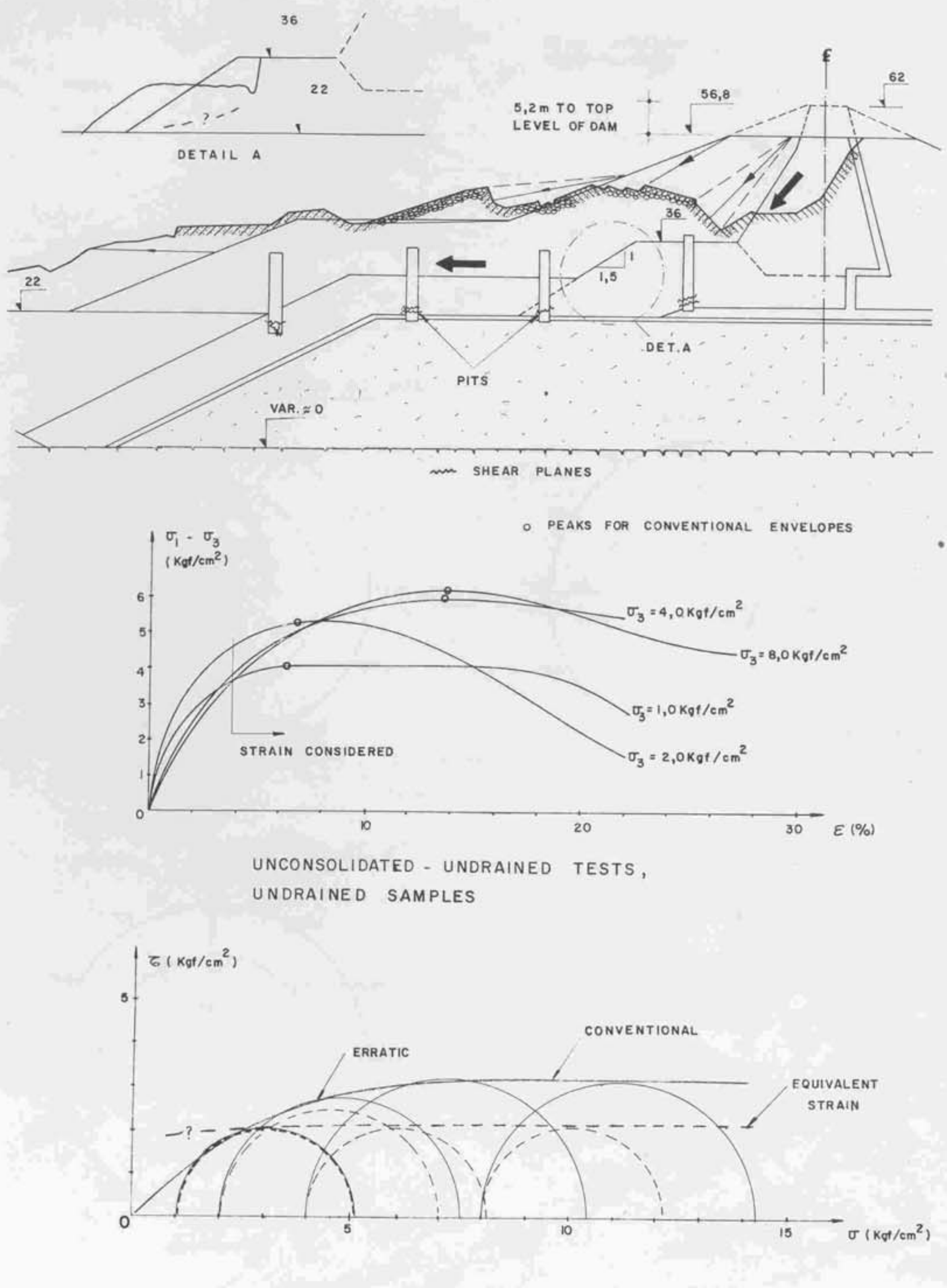
Ironically, a letter yet discussing the insertion of pore pressures in the computer stability analyses had been written and posted the day before the failure.

The conditions of the failure were duly identified both by the surface evidence of the kinematics (fig. 5), and by the careful identifications of orientations of planes (shear planes and lift layering) in inspection pits, duly plotted in stereograms (fig. 6). Surprisingly, despite the early warning on construction period sliding of the black clay, no piezometers had been installed to accompany pore pressure development ; a first piezometer had been installed but a few days before the major failure, but no readings had been taken.

During the excavation of the inspection pits, many of the layers were detected to be intensely laminated by overcompaction. Further, in the

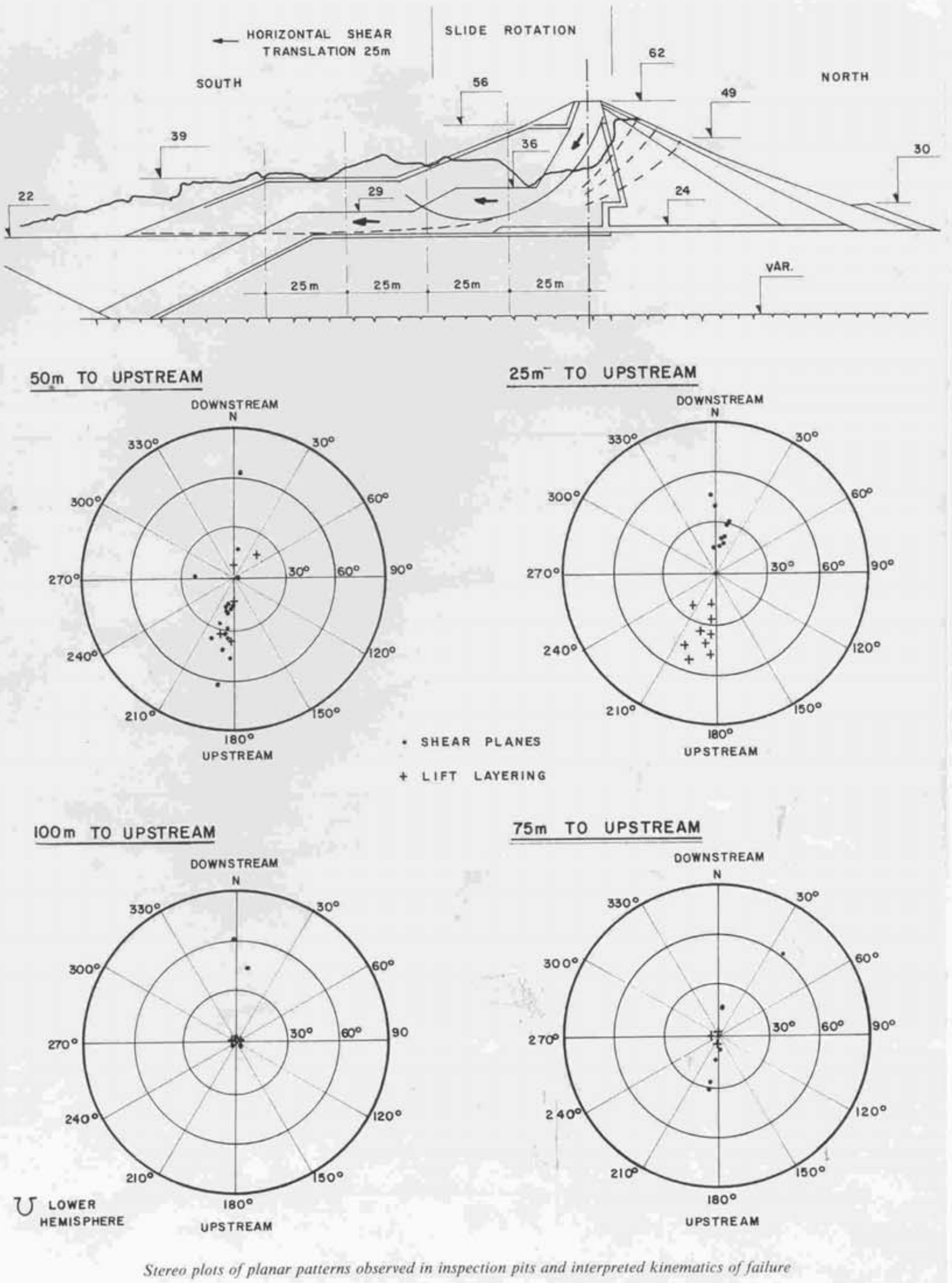
General view of the major construction period slide

PHOTO 1.



Failure surface and section. Triaxial tests and interpretation

FIG. 5.



Stereo plots of planar patterns observed in inspection pits and interpreted kinematics of failure

FIG. 6.

Looking upstream from the remainder of crest, down on rotated mass

PHOTO 2.

pits in the upper part of the slide mass considerable tension cracking was evident. Analyses are yet being performed to assess the degree of interference of such tension cracks, and so also of some proportion of the horizontal plane reduced to ϕ' values closer to the residual value.

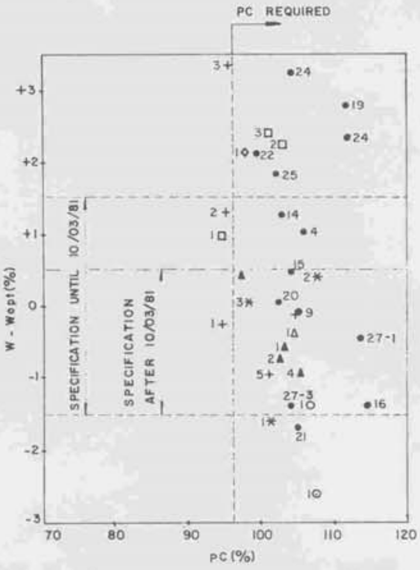
Although the earthmoving volume involved is of the order of a million cubic meters, corrective reconstruction was rapidly initiated and is expected not to retard planned impoundment.

There had been much attrition regarding apparent héterogeneities in the inspection testing. Many a geotechnician loses perspective of true problems because of the diminutive scales of routine laboratory tests. At the scale of tens of centimeters, indeed the black clay is very difficult to work in the field to the appearance of a homogeneous mass. However, all three slides were so perfectly homogeneous, didactic, and bidimensional, that absolutely no question can persist of unsatisfactory homogeneization. There can hardly be so well defined a failure except when a theoretical (design) aspect is intrinsically wrong. At any rate, for the purpose of record some index test data determined on small specimens in the inspection pits are summarized graphically in figure 7. In the same figure the frequency distributions of the inspection tests are also drawn. It can be seen that the statistical dispersions are indeed very small, noticeably so in comparison with some very important dams published (e.g. El Infiernillo, Presidente Aleman, Esmeralda-Chivor) picked at random.

QUESTIONS AND LESSONS

In a recent paper entitled " Practice, Precedents, Principles, Problems and Prudence in Embankment Dam Engineering " [de Mello, 1980] emphasis was placed on prudence as indispensable whenever an unusual situation seems to be at hand. It need hardly be repeated that the completely unexplainable point in this case history is how a textbook problem of classical soil mechanics was permitted to drag through a full year without any satisfactory and demonstrably confident solution. Administrative action could have well averted, or compensated for, the technical failings. Throughout the paper, key points in error have been mentioned, but they are so many that it would seem important to draw a more concise lesson.

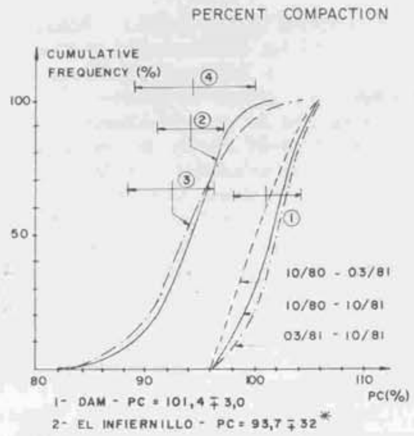
Conventional rules, simplified, insufficiently understood, but/and therefore used with unwarranted faith, may be the general criticism. Straight-line shear strength equations, extrapolated with no consideration to limitations ; in the



INDEX TEST DATA DETERMINED IN THE INSPECTION PITS AND TRENCHES

SAMPLE	PIT ●	STAKE	LEVEL (m)	DISTANCE FROM THE AXIS (m)
2	4	45+10	27,8	50 / UP
1	9	49+10	35,4	0
2	14	57+00	27,0	25/US
3	15	57+00	26,2	50/US
3	16	57+00	25,7	75/US
1	19	60+15	26,5	50/US
1	20	64+10	27,0	25/US
1	21	43+10	32,0	50/US
1	22	45+10	22,3	75/US
1	23	53+10	19,0	100/US
2	24	53+05	22,3	100/US
3	25	57+00	24,5	100/US
1	27	54+10	28,4	50/US
3			23,0	

SAMPLE	TRENCH	STAKE	LEVEL (m)	DISTANCE FROM THE AXIS (m)
1	1-▲	39+10	56,2	3/DS
2			55,1	3/DS
3			54,5	3/DS
4			52,2	2/DS
1	2-+	41+10	55,7	3/DS
2			55,0	3/DS
3			53,2	4/DS
4			52,3	4/DS
5			51,3	2/DS
1	3-✱	43+10	55,1	2/DS
2			53,8	3/DS
3			53,4	4/DS
4			52,4	2/DS
1	4-○	63+10	55,7	1/DS
1	5-△	65+10	55,8	2/DS
1	6-□	63+10	56,5	2/DS
2		46+10	52,0	2/DS
3		46+10	48,4	2/DS
1	7-◇	45+10	51,1	3/DS
1	12⊙	64+10	48,7	2/DS



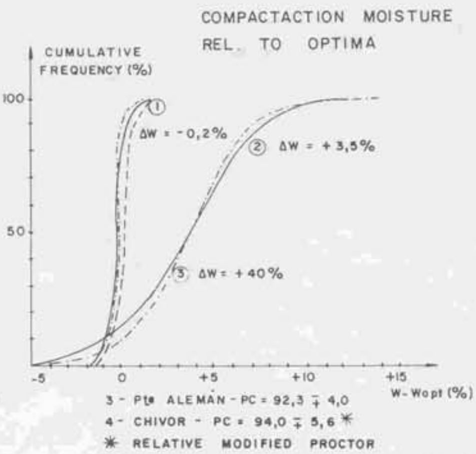


FIG. 7.

Compaction control index test data compared with specifications and dispersions compared with important dams

Views of slide scarp with stretches broken off in tension and areas slickensided in shear
PHOTO 3.

Close-up of black clay partly slickensided in shear, partly broken off in "granular chunks"
PHOTO 4.

Tension cracks retrogressing behind slide scarp on remainder of crest and into upper part of downstream slope
PHOTO 5.

case of an extensive potential shear surface in a preferentially weak material, the questionable use of a Mohr envelope of maxima under different confining stresses irrespective of such maxima occurring at noticeably differentiated strains. Imposed circular failure sliding surfaces, not permitted to select the critical preferentially weak composite surface; further, in the case of an extensive surface, the questionable implicit assumption of rigid-body movement, rigid-plastic behavior presumed at coincident failure strains. Inspection heavily based on index tests, insufficient and often unsatisfactory portrayals, rather than requiring the development of a visual-tactile feel of features (e.g. laminations etc.) often capable of being much more significant and detrimental.

How many of the geotechnicians dedicated to eager study of publications and to intimate communication with equations, computers and laboratory tests, would be preserved from treading a similar failure path in gallant faith? It is sad that we should, in geotechnical engineering, have so much to learn from failures. It would be sadder that we should have to relearn from additional failures what could have been claimed as known.

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