INQUIRY INTO ROCKFILL SLOPE STABILITY ANALYSES

Victor F.B. de Mello* L.S.A. Kaimoto** Makoto Namba**

INTRODUCTION

The use of compacted rockfills for dams opened immense technical and economic advantages, growingly recognized; the well proven benefit has been of greatly reducing the unfavourable deformabilities of dumped rockfill. However, collateral improvements of, shear strength, intuitively expected, have not been convincingly demonstrated or used; on the contrary, because of misplaced geotechnical theory of infinite slope stability of cohesionless soils related to triaxial compression tests, the slopes empirically accepted for end-dumped rockfill most often have been flattened for the respective compacted rockfills treated as presumed big-size "sands", even though there were no reported or documented failures in the end-dumped natural slope rockfills, even in high seismicity areas.

In a paper that set a milestone in the evaluation of rock fills by soils mechanicians. Leps[15] reviewed questions on the shear strength of rockfill, emphasizing the "daring" nature of design of rockfill dams based "not on diagnostic testing of the strength of rockfill but on the satisfactory performance of many prototype fills". Analysing the available big-specimen triaxial shear test data he interpreted the significant variation of "friction angle" with pressure as roughly linear, and focussed importance on the "average friction value" at very low pressures, whereby low (9-17 m) "unconventional dams" of 1 on 0.5 slopes stood perfectly, making it "necessary and justified to review both the stability analysis procedures, and one's assumptions regarding rockfill strength", pointing out as an example that "the infinite slope analysis method is inapplicable".

Notwithstanding the impression that the comments applied only to low dams, and that failure modes other than sliding shear of a limit equilibrium mass might have to be formulated, the more recent trend towards acceptance of steeper slopes in high compacted rockfill has begun to advance: but the profession is tied to very slow progress, because of the difficulty and cost of appropriate testing of rockfill as constructed, coupled with limited mental models for analysis and confident extrapolation of prototype experience, and also because of the slow build-up of such experience.

Moreover, the acutely increased consciousness of seismic risks to dams during the past decade has renewed concern with prospective deformabilities under corresponding conservative cyclic loadings; thus, the profession must face with concern the extremely low probability with which rockfill dams subjected to strong seismic shaking will chance to furnish experience of use in favouring steep design slopes.

It is this paper's intent to analyse critically the subject of compacted rockfill slope stability. As a premises there is the historic witness that in major scientific fields many advances quite beyond the evidences that nurture empiricism, have been formulated purely by reasoning, i.e., mental experimentation; thus to restrict dam design progress to lessons from prototype observations would constitute an unacceptable stand. Incidentally, any present analyses/predictions are no more than exercises of such mental experimentation, and, therefore, the authors' proposition represents no more than the offer of an additional different, mental model, hopefully better.

The paper submits the certainty that much steeper slopes than presently used should prove satisfactory for questions of sliding stability. Moreover, that in facing problems of deformability under strong earthquake shaking, in lieu of postulating additional methods of predicting via small scale test parameters and computations, a far more promising avenue may be sought through proper treatment of the rockfill, if the mental model be judged acceptable, for orienting such treatment and the collection of field data indicative of

* Professor (Retired), Sc.D. (M.I.T.), University of Sao Paulo, Consulting Engineer.

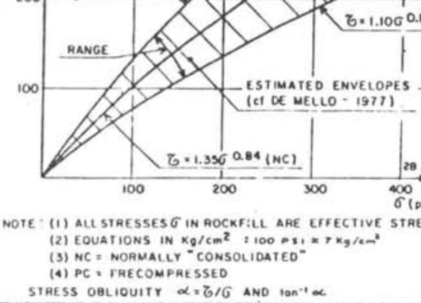
** Engineers, Victor F.B. de Mello & Associates, Consulting Civil-Geotechnical Engineers, R. das Madressilvas 43, Brooklin, 04704-070, Sao Paulo, Brazil.

concomitantly benefitted secondary behaviour.

There have been so many partial demonstrations of this postulation in recent publications (viz. Charles a Soares,[3]) that, on the one hand, one risks not attributing each advance to its real originator, and on the other hand it appears indispensable to attempt coordinating past, present and future mental modelling on this important design feature. The authors apologies in advance for any unintended lapse on the first count. On the second, the authors offer their thoughts as representing hypotheses, presumed very fruitful, to be debated.

SHEAR STRENGTH ENVELOPES ATTRIBUTED TO UNIFORM ROCKFILL

Many series of special laboratory tests have established the curved Mohr strength envelope applicable to sound angular aggregate intended to model, in laboratory specimens, the behaviour of rockfills from different quarries. Recent special laboratory tests (e.g. Charles and Watts,[4]; Barton and Kjaernsli, 1, etc.) have shown that at very low confining stresses the curved envelopes are even steeper than earlier suggested (e.g. de Mello,[9]); however, this incremental influence in support of steeper slopes is herein set aside in comparison with other factors demonstrably more significant. For the present analyses, the strength envelope used is shown in Figure 1, collated from the triaxial compression test data analysed in 1976 (de Mello,[9]).



NOTE : (1) ALL STRESSES σ IN ROCKFILL ARE EFFECTIVE STRESSES
(2) EQUATIONS IN Kg/cm^2 : 100 PSI ≅ 7 Kg/cm^2
(3) NC = NORMALLY "CONSOLIDATED"
(4) PC = PRECOMPRESSED

STRESS OBLIQUITY $\alpha = \tau/\sigma$ AND $\tan^{-1}\alpha$

σ (psi)	MATERIAL (*) A		B		C		D		NC $1.35\sigma^{0.84}$		PC** $\sigma_p = 100$ psi		PC** $\sigma_p = 210$ psi	
	α	$\tan^{-1}\alpha$	α	$\tan^{-1}\alpha$	α	$\tan^{-1}\alpha$	α	$\tan^{-1}\alpha$	α	$\tan^{-1}\alpha$	α	$\tan^{-1}\alpha$	α	$\tan^{-1}\alpha$
14.2	1.05	46°	1.50	56°	1.65	59°	1.75	60°	1.35	53°	2.40	67°	2.75	70°
56.9	0.75	37°	0.93	43°	1.18	50°	1.40	54°	1.08	47°	1.29	52°	1.73	60°

(*) cf CHARLES AND WATTS, 1980
(**) cf FIG 2

FIGURE 1: Basic Strength Equations Used NC

For the sake of comparison, some stress obliquities τ/σ that represent the higher strengths determined by Charles and Watts[4] are tabulated alongside with those herein used and would only reinforce the points submitted herein. However, they were not used because they probably include some recompression effects from the specimen compaction and there is no way of confirming this hypothesis and accounting for it because the nominal precompression pressures from semilog compressibility plots are not available.

Besides the curvature of the strength envelope it is postulated that one principal factor affecting stress-strain-strength behaviour of clean angular aggregate and rockfill is the effect of precompression, by crushing of point contacts. In so-called cohesionless materials the slope stability is mostly dependent on the strength envelope in the low stress range Leps[15], de Mello,[7], Charles, et.al.,[4], etc.). The influences of precompression, routinely correlated with nominal "over-consolidation ratios" OCR, are increasingly beneficial at the high OCR values that pertain to the low stresses. Under prestressed conditions the start of the strength envelope will be considerably steeper, and even suggest an apparent cohesion intercept, thus favouring the slope stability additionally.

Special laboratory tests have given definite evidence (e.g. Veiga Pinto,[22]) of the anticipated benefits of precompressions in clean angular aggregate used for modelling rockfill. The principal interest in this paper is directed towards analogous angular quarried rockfills, and extending the experience derived from end-dumped rockfill slopes to stability analyses presumed applicable to corresponding compacted rockfills. Based on some field experience (e.g. de Mello,[6,7]) one assumption used for the present computations is that the routine four passes of the 10-ton vibratory roller on rockfill lifts about 0.8 to 1.0 m thick, produce a compaction precompression of about 7 kg/cm^2. The corresponding benefit on strength envelope has been adjusted by analogy from the laboratory test data on crushed aggregate (Veiga Pinto,[22]) as indicated in Figure 2. Hereinafter the comparative conditions analysed are subdivided into the normally consolidated (compressed) case, NC and the precompressed case PC, with values OCR > 1.

Benefits of precompression derive from (de Mello,[7]) the compression-expansive hysteresis attendant to the crushing of rock-rock point contacts, and should be of importance far beyond the mere improvement of testing techniques (Charles and Watts,[4]) in conventional trixial compression tests that apply increasing deviator stresses to specimens starting from a non-prestressed condition. Some collaries ensue from the minute strain recovery of the crushed points upon stress release, representing a very high nominal "modulus of elasticity" ($E_{\Sigma\infty}$).

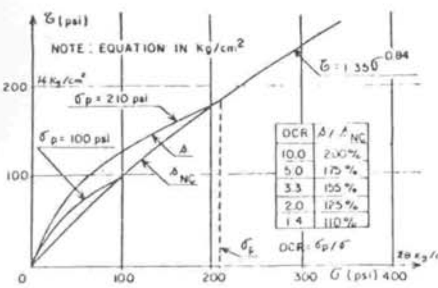


FIGURE 2: Strength Envelopes Adopted for Pre-Compressed Conditions OCR > 1

Firstly, one reasons that the traditional emphasis of compaction as the achievement of a given higher density, has been a gross mistake, and consequent deterrent to progress in the subject of stress-strain-strength behaviour of rock and crushed aggregate: an infinitesimal change of overall dimensions and density can correspond to a big change in stress and areas of angular point contacts, and density is too crude an index for the purpose, besides being too erotically variable as a function of grainsize distributions within the modest test volumes.

Secondly, one must concede that the locked-in horizontal compaction stress must be released readily as the roller moves away, the principal horizontal consequence of compaction being on lateral deformations and Poisson's ratio , and not on slope stability; thus one first sets aside, for conservative simplification, the smaller beneficial influence of the inversion of maximum and minimum principal stresses up to a certain stress level, when over-burden stresses on the lift generate deviator stresses with regard to the residual confining stresses from compaction (de Mello,[9]).

Moreover, since for decreased compressibility one modern trend has been to accept, and even prefer, the use of broadly-graded "dirty" rockfill, it must be emphasized that in such cases the improved behaviour is dependent on reduced porosity from denser packing, and on multitudinous grain-to-grain contacts at moderate stress, and thereby the salient hysteresis of crushed point contacts of clean angular rockfill is excluded. Thus the case of dirty rockfills, with attenuated benefits of precompression, will have to be considered in a separate paper.

Some authoritative opinions propose using vibratory rollers of greater static and impact stresses for the higher rockfill dams, in order to increase the σp stresses. For possible comparisons of anticipated benefits, the principal computations are repeated for a hypothetical σp of 15 kg/cm², and assuming effects directly related to the σp stresses and the OCR ratios relative to each maximum precompression stress.

Finally, in the postulated "instantaneously crushable" material one must recognize that in general there will be two distinct conditions causing precompression : compaction precompression σ p and the over-burden precompression γ p and the over-burden precompression γ_p γ z. In most of the computations the vertical stress γ z at the bottom of each vertical column of rockfill was adopted as an acceptable simplified approximation of the stress controlling overburden prestressing. De Mello[6,7] has emphasized the need to include influence value I of stress transmissions due to the overlying trapezoid in order to improve assessment of σ v values, but this correction as further affecting slope stability computations is not included herein.

If the mental model of the importance of precompression be accepted, one will promptly see the need to extend judicious laboratory testing into OCR conditions related to σ p stresses caused by impact contact crushing, and detected, in nominal values, by conventional ΔH vs log σ plots, or other appropriate techniques.

CONVENTIONAL SLOPE STABILITY ANALYSES CONDUCTED, ROCKFILL AS MODERNLY ACCEPTED

Procedures for computing factors of safety of slopes have been advanced to a point where different methods can be used with no more than a second-order difference in computed result (viz. Fredlund,[11]). The present intent is to raise comparative points affecting present thinking on rockfill slope stability, with no intent to indicate preference for one method over another. Most of the analysis were carried out by Janbu's Generalized Method (Janbu,[13]), and some of the cases were checked by Sarma's method (Sarma,[18]) both for the sake of illustrating the similarity of results, and for extracting indications of interest for discussing seismic stability. All analysis run and presented merely discuss and exemplify concepts and trends through typical comparative numerical values.

One early dogma of conventional soil mechanics would establish the factor of safety of a slope is of purely cohesionless material of friction angle ϕ, by the equation $F = \tan \phi / \tan i$, derived from the sliding-block mental model of elementary physics. This was shown to be devoid of engineering interest (de Mello,[9]) because it concerns the sliding of an infinitesimal volume $\Delta V \rightarrow dV \rightarrow 0$ on the surface of the slope. At any rate, one concludes that attention must concentrate on small volumes and shallow sliding surfaces (surface raveling, theoretical infinite radius), as well as on the rate of change of strength and sliding stability with depth of sliding surface.

For a start the 1.0V:1. 3H slope was adopted, which is routinely taken as the "angle of repose" of dumped rockfills. For minimizing "end effects" a high slope was taken, 200 m as an example. Figure 3 summarizes analysis by Janbu's method, of six slip surfaces. The critical depth, for the "normally consolidated curved envelope" (NC envelope), would reach a maximum depth of about 50 m, with a Factor of Safety F = 1.43. It is seen that with a curved strength envelope conventional slip surface analysis do reach finite critical depths (cf. de Mello,[9], Costa Filho and Thomaz,[5]) obviating the indeterminate condition attendant on the oversimplified linear cohesionless strength equation conventional for "ideal" sands.

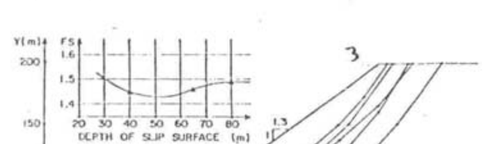


FIGURE 3: Routine Stability Analysis. NC Curved Equation, Janbu, Basic Case

In cases of engineering structures that have recorded experience with prototype failures it is not altogether difficult to assess the significance of specific numerical values of F, and corresponding minimum required values for comfort. In the case of rockfills there are no known failures from which to draw lessons by back analysis, and, therefore one must investigate trends of rate of change, of causes vs. effects, for assessing the numerical levels of F to adopt as satisfactory. Three factors surface as significant:

1. One is the rate of change of F with varying position of the slip surface, suggesting if the position of critical limit equilibrium lies within a tight band, concentrating higher shear strains;

2. Another is the rate of change of sliding volumes (consequent damage) with change of F;

3. The third comprises the rate of change of the above parameters with change of slope, since in case of discomfort the present direct remedy would be to flatten the slope.

Some indications on such questions derive from repeating the analysis for slopes increasingly steeper. The results are plotted in Figures 5(a), (b) and (c). One concludes that under the average NC strength envelope adopted, the sliding failure (F < 1.0) would occur with a slope almost as steep as 1.0V:0.7H. In graph 5(a) one sees that the maximum depth of the critical surface ranges between 40 and 50 m, with little variation for the different slopes analysed. Moreover, graphs 5(b) and (c) indicate, in two different manners, that as the critical slip volumes increase, the minimum F value also increases noticeably.

Geotechnical engineering tradition has commonly adopted constant values $30° < \phi < 45°$, for slope stability F analysis in sands, gravels and rockfills. For the curved strength envelopes F depends on stress levels and depth of critical surfaces. Figure 4(a) indicates the gain in Factor of Safety, F, of the critical surface for the adopted curved envelopes relative to typical limiting constant ϕ values. Claims in favour of the pre-1960 conventional adoption of $\phi = 45°$ would err with excessive pessimism for shallow surfaces, and in the opposite direction for eventual deep surfaces under high stresses, which dominated the concerns of the score of years (1960-80) after large-diameter triaxial testing of crushed rocks began, for high compacted earth-rock dams. The case of eventual deep-seated sliding in earth-rock dams requires further study in order to obviate false conclusions. In Figure 5(a) "case 3" shows that the simplified early design precedent using $\phi = 45°$ for rock becomes increasingly unsatisfactory with increased height of dams and depth h [Figure 4(a)] of critical slip surface.

In Figure 4(b) the results explore the statistical dispersion of strength envelopes of Figure 1 for the basic 1.0V:1.3H. F varies ± 25% around the average. These indications are compatible with the frequency distribution data of end-dumped and bottom-excavated rockfill stockpiles (de Mello,[8]) wherein the limiting observed obliquities (directly related to stability) vary between (29 ± 14) % higher than the average and (30 ± 3) % lower than the average.

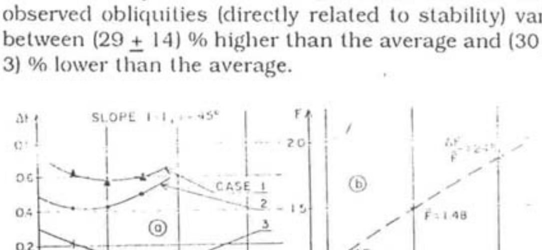


FIGURE 4: Range of Variation of Conventional F Values Compared with Classical Assumptions

SOME CONSIDERATIONS AFFECTING UPSTREAM SLOPES OF EARTH-ROCK DAMS

For upstream shells of earth-rock dams there are

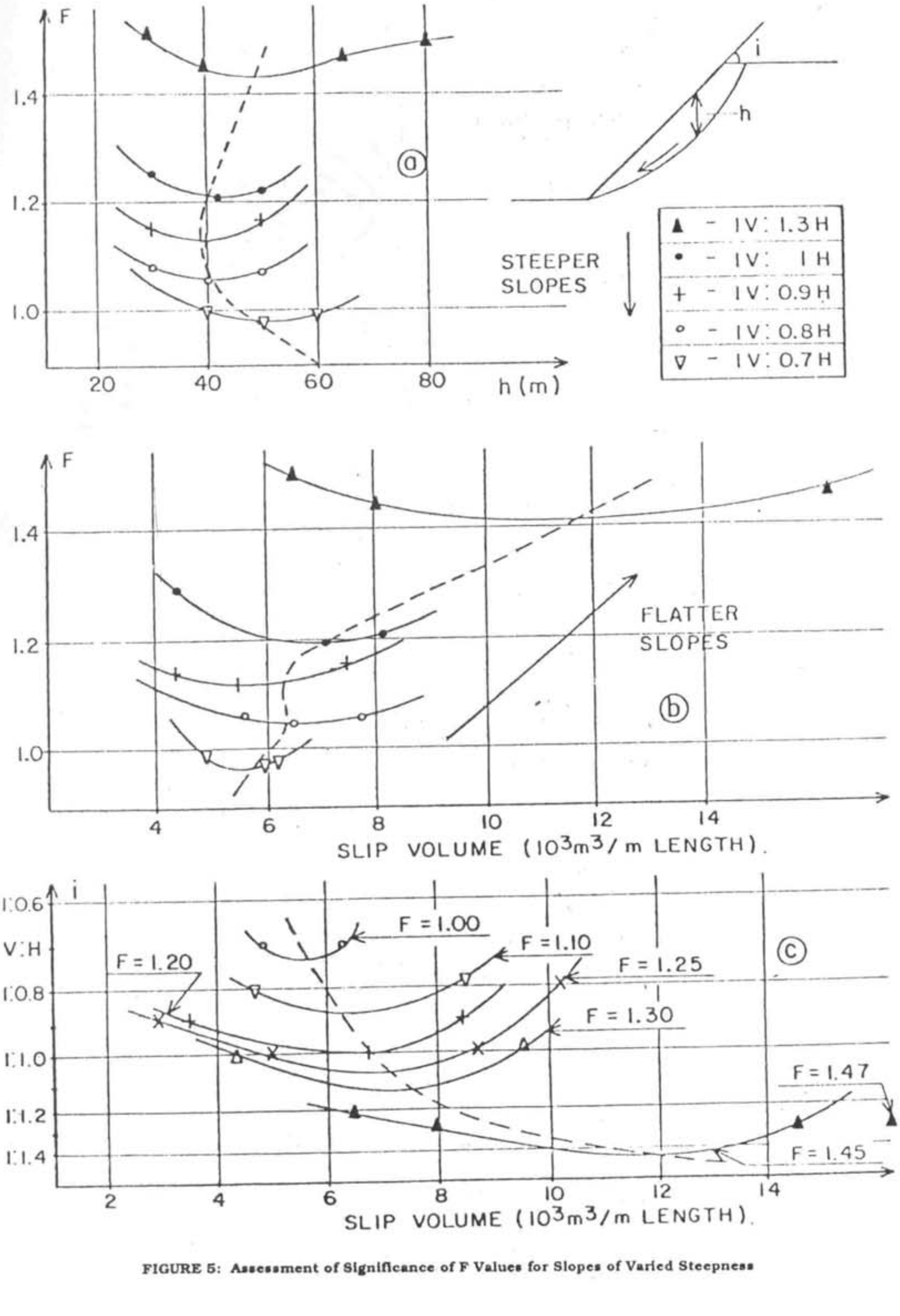


FIGURE 5: Assessment of Significance of F Values for Slopes of Varied Steepness

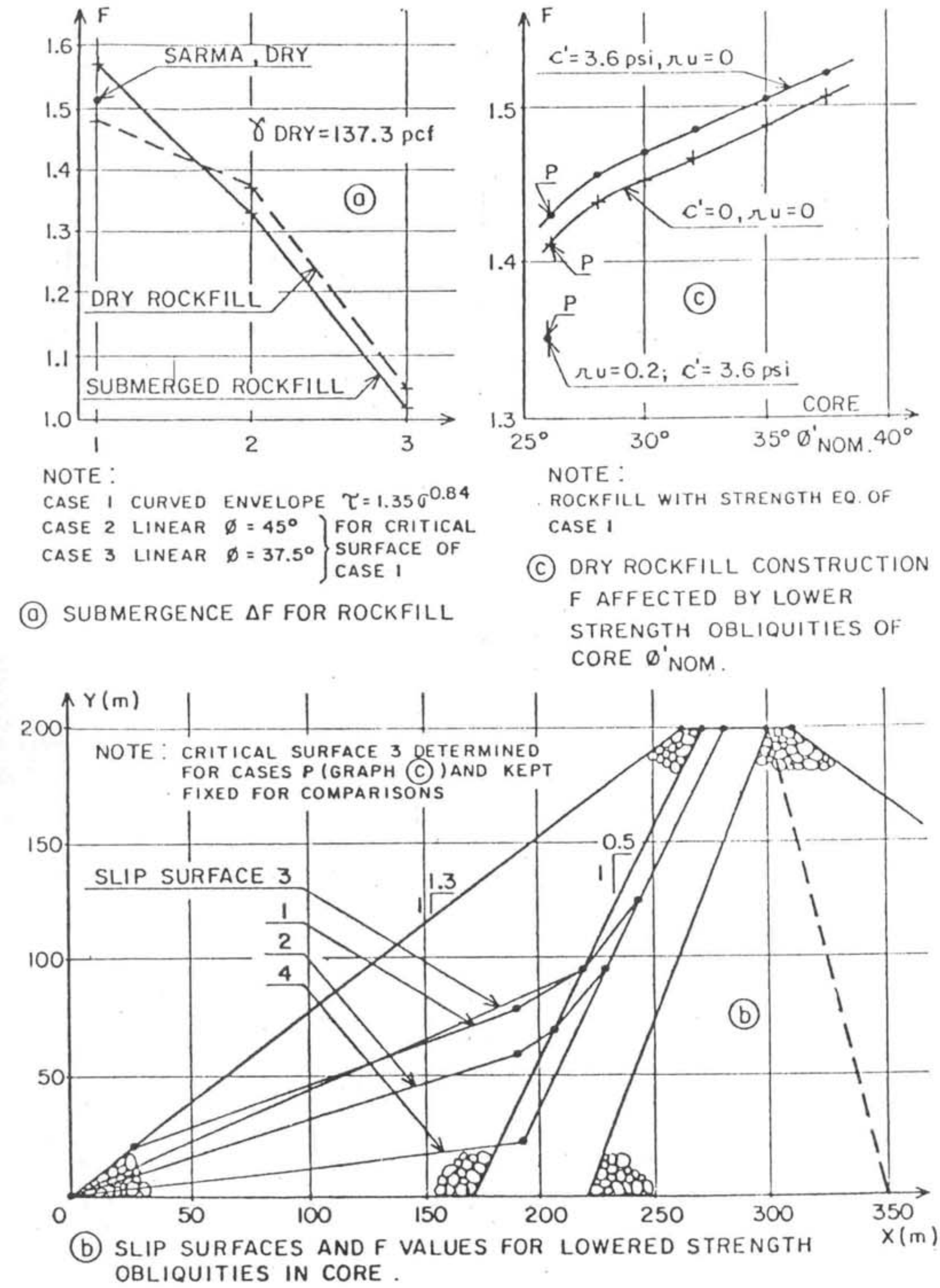


FIGURE 6: Some Cases of Upstream Rockfill of Earth Rock Dams, NC Condition

so many intervening factors in any given project, especially when involving variabilities of clayey cores, that one cannot attempt any more than to illustrate concepts by a couple of examples.

Under simplest idealizations, the least effect to incorporate in the stability analysis of the rockfill must include submergence in reducing its normal affective stresses. The basic rockfill strength equation is assumed unaltered by wetting and/or incremental crushing of contacts (and consequent further reduced intergranular stresses), an assumption to be checked by laboratory tests, and expected to be different for different rock types. Moreover, it was conservatively assumed that the precompression and OCR induced by submergence do not increase shear strength by hysteresis: the stress release was followed along exactly the same strength equation as during stress increase.

Since the changes ΔF were anticipated to be small, this case was employed simultaneously for conducting comparative analysis by the Janbu[13] and Sarma[18] methods for the curved envelope, and also by the Janbu method for linear envelopes of $\phi = 37°.5$ and 45°. Figure 6(a) summarizes results for the 200 m slope of 1.0V:1.3H. The critical surface's F = 1.48 for the dry rockfill increased by ΔF = 0.9 to F = 1.57 due to submergence to elev. 180 m. The corresponding Sarma static analysis, dry rockfill, gave F = 1.51, close enough to the Janbu 1.48 so that most subsequent analysis were run by the Janbu method, except when including seismic effects, for which Sarma's method is convenient. The two cases analysed for constant ϕ indicate the imprecisions in computations (2 to 5%), since firstly the results on a linear strength envelope should not evince any difference due to submergence, and secondly, the F values for the 1.0V:1.3H slope (case 3, Figure 6(a) are close to the theoretical F = 1.0 corresponding to $i \simeq \phi$.

The important interference on slope stability of upstream zones derives from the weaker core material in conditioning more critical surfaces (cf. Sultan and Seed[20]). Although the topic lies outside the scope of this study, a few simplified runs by Janbu, Figure 6(b), using judicious parameters for the core, help to indicate its influence on the construction period stability. For simplicity, any combination of values c', ϕ' and pore pressure coefficient ru can be transformed into critical shear strength obliquities ∞ crit. simulating nominal ϕ' nom. values for the core. The analysis used this expedient. The results for a typical sloping core position, Figure 6(c), show a modest drop in F values, because of the depth of the critical surface, and still leave quite an acceptable upstream stability.

For the upstream shell, assuming the NC critical surface within the rockfill itself [Figure 3, Figure 6(a)], the comparative F values were also analysed for confirming the benefits of compaction precompression. The results are tabulated below. The submergence was now taken with strength values along the PC envelopes. For simplicity and conservatism, in the case of $\sigma p = 7$ kg/cm^2 where some slices do reach $\sigma v > \sigma p$, the submergence stress release neglected further benefits of precompression to $\sigma\, p\, \gamma z$ of the dry rockfill; at $\sigma\, v > \gamma\, p$ only the curved NC envelope was used. It is demonstrated that under reservoir submergence the rockfill slope profits from conditions much less critical than during fill rise, that is, with stability greater than that of "pretested" stable conditions.

MORE DETAILED ANALYSIS OF THE DRY ROCKFILL SLOPE FAILURE CONDITIONS, NO COMPACTION PRECOMPRESSION INCLUDED

Firstly, for the NC condition of dry rockfill, 200 m slope, the possible condition and significance of any slip failure were investigated by using increasingly steeper slopes. The critical surfaces and F values were meticulously investigated by Janbu, giving the plotted curve of Figure 7(a). Failure would occur with a slope of about 1.0V:0.7H, i.e., i = 55°. A check by the Sarma analysis gave exactly the same result. The modest increase of F values with significantly flattened slope should temper our judgement regarding required F values: in materials of shear strength dominantly based on friction, statistical dispersions, that call for higher F values, are relatively small.

Further, using the Sarma analysis, the distributions of normal and shear stresses [Figures 7(b), (c)] were computed along the rigid block sliding surface, and consequently the stress obliquities were derived [Figures 7(d)] showing local concentrated failure conditions at the top. The singular computed critical stresses of the top of Figure 7(b), (c), (d) are physically impossible, and might relate to the opening of a crack. The computer program is being adjusted to reject such singularities, seeking revised reruns. Such future local revisions should not alter present conclusions.

PURSUING THE SIGNIFICANCE OF EVENTUAL FAILURE OF A SLOPE TOO STEEP

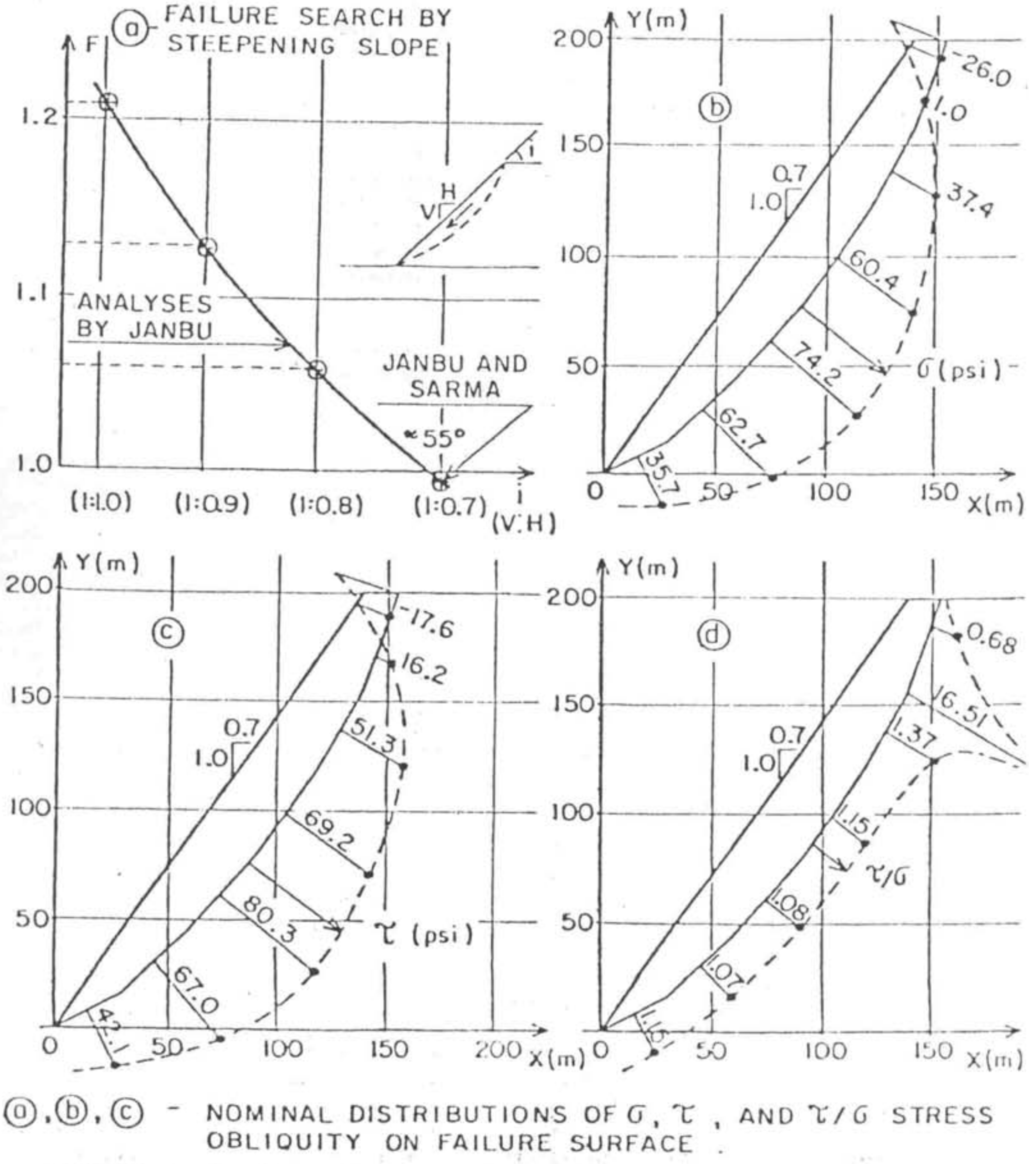
It was established above that the 55° NC dry rockfill slope would just fail (F = 0.98). The critical slip surface, 1, (by Janbu) is shown in Figure 8. It was further analysed for assessing the significance of the failure possibility, and its tendency to progress or to stabilize. The principal hypothetical slip surfaces are also shown in Figure 8 (and details M, N).

One manifest basis for such assessment calls for inclusion of precompressions, both from compaction, and those automatically generated by overburden stresses of effects rapid enough to be accepted as nominally instantaneous. It is argued that a limit analysis conducted routinely has no meaning: any such analysis constitutes firstly an inquiry into an eventual "tendency to fail", and its significance depends on the

UPSTREAM SHELL, 1.0V:1.3H, JANBU ANALYSES FOR COMPARISONS
ON CRITICAL SURFACE OF NC CASE.

DRY ROCKFILL	NC $F_o = 1.48$*	$\sigma_p = 7 kg/cm^2$	$F = 1.65$	$\Delta F/F_o = 11.5\%$	
		$\sigma_p = 15$	$F = 2.30$	$= 55\%$	
SUBMERGED BY 180m OF WATER	NC $F_1 = 1.57$ $\Delta F/F_o = 8.3\%$	$\sigma_p = 7$	$F = 2.27$	$= 53\%$	$\Delta F/F_1 = 45\%$
		$\sigma_p = 15$	$F = 2.97$	$= 101\%$	89%

*Check by SARMA, $F_o' = 1.51$ (2% higher)



(a),(b),(c) - NOMINAL DISTRIBUTIONS OF σ, τ, AND τ/σ STRESS OBLIQUITY ON FAILURE SURFACE.

FIGURE 7: Closer Investigation of Slip Failure Surface and Conditions, Dry, NC, Curved Envelope

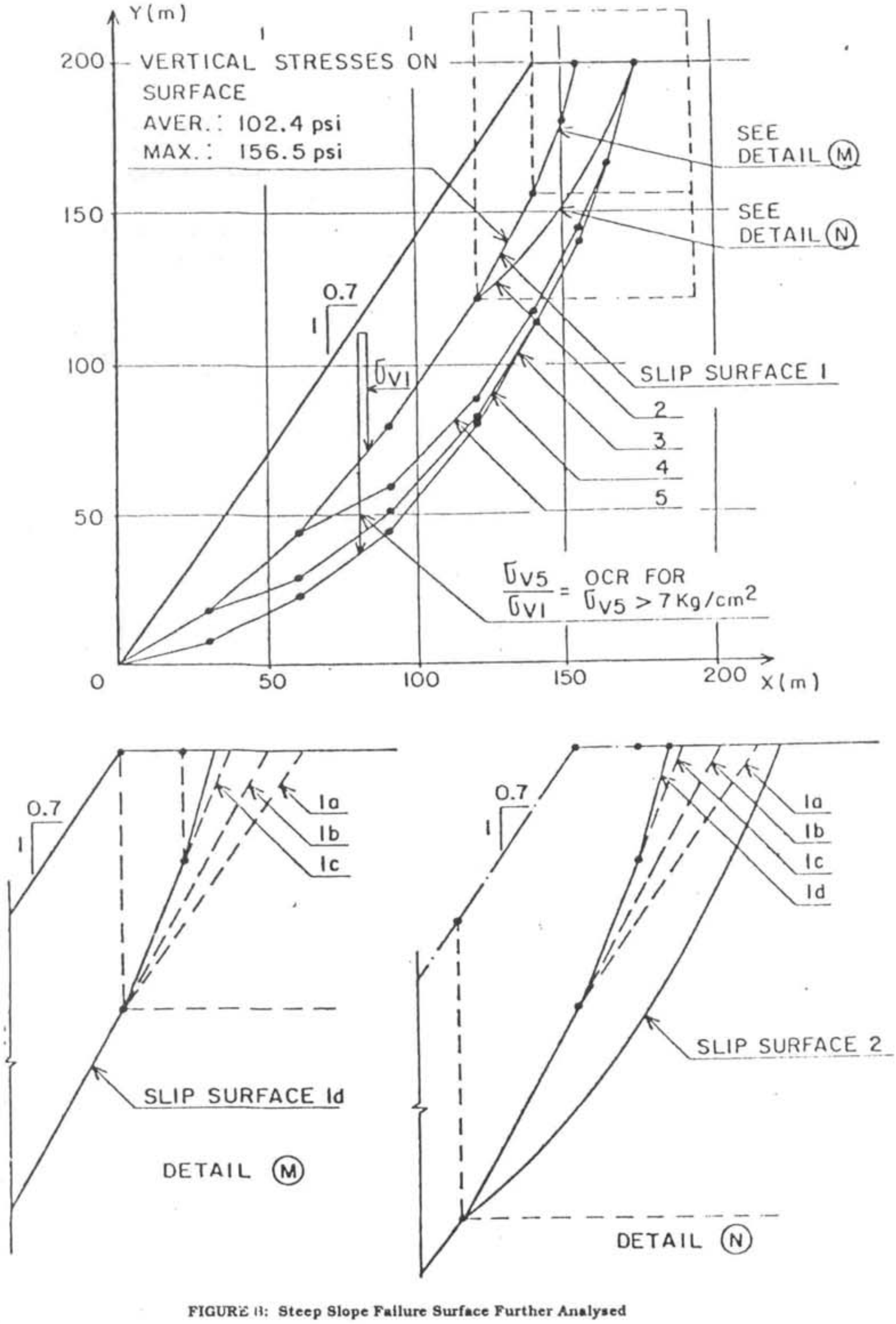


FIGURE 8: Steep Slope Failure Surface Further Analysed

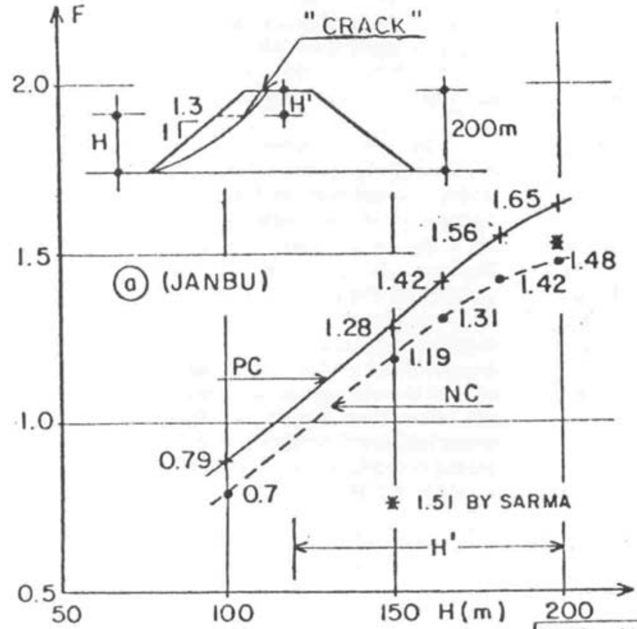
TABLE I

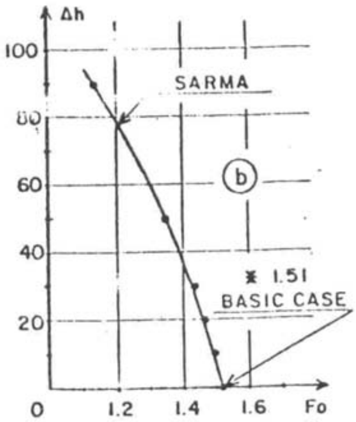
RESULTS OF ANALYSES AS PER FIGURE 8

SURFACES	CONDITION		F	$\Delta F/F\%$	COMMENTS	
1	NC DRY, i.e. $\sigma_p = 0$		0.98		Uncompacted rockfill general critical surface defined. Maintained for subsequent comparisons	(A)
1	$\sigma_p = 7$ kg/cm^2 (100psi)	NC used for $\sigma_v > \sigma_p$	1.05	7	Compacted rockfill. Assuming eventual slip during load increase.	(B)
1	$\sigma_p = 7$ kg/cm^2 (100psi)	PC used for $\sigma_v > \sigma_p$	1.47	50	Assuming unloading condition resulting from 10m drop of slip crest, and NC accumulated toe wedge of equiv. vol.	(C)
1	(213psi) $\sigma_p = 15$ kg/cm^2 HYPOTHET.	NC used for $\sigma_v > \sigma_p$ (inexiscent)	1.68	71	Max. σ_v on slip surface = 156psi, i.e. < σ_{pc}	(D)
1		PC used	1.68	71	Entire slip surface under compaction σ_p conditions.	
1a 1b 1c 1d	Overall slip surface F affected by steeper upper (43m) stretch, NC, DRY (DETAIL M)		1.0 0.99 0.98 0.98		Except for favorable hetero-geneities, wedges of steeper upper stretch will topple.	(E)

SURFACES	CONDITION		F	$\Delta F/F\%$	COMMENTS	
2	Partial slip upper (87m) (DETAIL N)	1.a	2.18		Compaction precompression essentially precludes failure of shallow surfaces. Construction assumed to outer surfaces 1a, 1b, 1c, 1d.	(F)
2	Stretch under slip surface:	1.b	2.14			
2	PC, $\sigma_p = 7$kg/cm^2	1.c	1.86			
2	(100psi)	1.d	1.75			

SURFACES	HYPOTHETICAL SLIP SURFACES UNDER SLIP SURFACE 1					REPEATED FOR SURFACES 4		5		
3	NC, DRY		1.03	$\Delta F/F\%$	Slope as built	1.07	$\Delta F/F\%$	1.10	$\Delta F/F\%$	(G)
3	$\sigma_p = 7$ kg/cm^2 (100psi)	NC for $\sigma_v > \sigma_p$	1.19	15.5	Under extreme hypothesis of "total removal" of sliding mass above surface 1, overburden stress release generating strength in PC envelope.	1.15	7.5	1.14	3.6	(H)
3	$\sigma_p = 7$ kg/cm^2 (100psi)	PC used for $\sigma_v > \sigma_p$	1.93	87		1.80	68	1.90	73	(I)
3	$\sigma_p = 15$ kg/cm^2 (213psi)	NC for $\sigma_v > \sigma_p$	1.41	37		1.45	36	1.52	38	(J)
3	$\sigma_p = 15$ kg/cm^2 (213psi)	PC used for $\sigma_v > \sigma_p$	1.99	93		1.92	79	2.17	97	(K)
3	NC, DRY $\sigma_v = \sigma_p$		1.84	79	Failure hypothesis after rockfill built to full height.	1.67	56	1.81	65	(L)

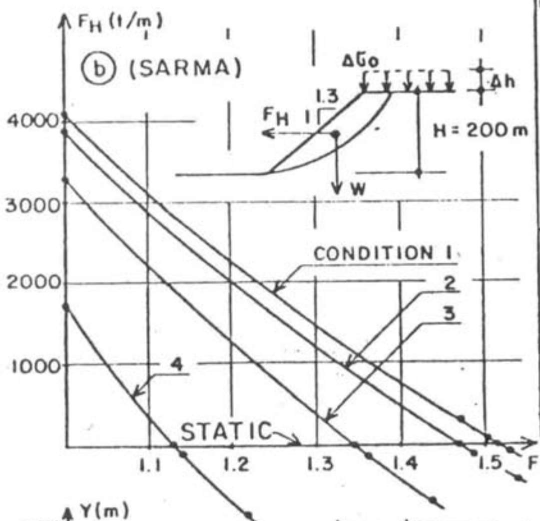


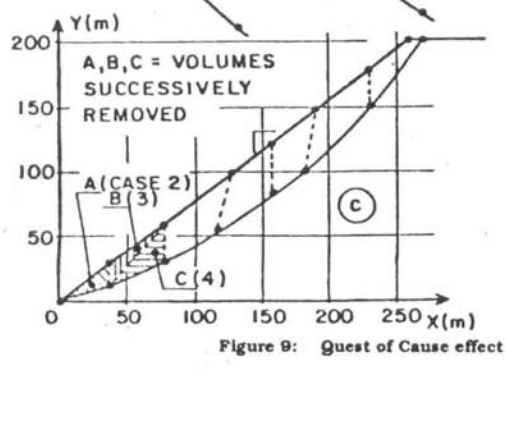


137 (pcf) ≡ 2.2 t/m^3

COND.		Δh_m	W (t/m)	K_H	K_V	F	γ (pcf)
1	b	0	17.52[illegible]	[illegible]	0	1.0	137
				0	0	1.52	137
2		20	18.[illegible]77	0.216	0.029	1.0	137
				0	0.029	1.47	137
3		50	18.885	0.174	0.070	1.0	137
				0	0.070	1.34	137
4		90	19.963	0.085	0.119	1.0	137
				0	0.119	1.14	137
5	b1	0	35.056	0.223	0	1.0	275
				0	0	1.43	275
6		0	52.584	0.133	0	1.0	412
				0	0	1.30	412
7		** 0	17.528	0.367	0	1.0	137
				0	0	1.72	137
8		0	17.528	0.748	0	1.0	137
				0	0	2.36	137

**·(5,6) INCREASED DENSITY SIMULATING VERTICAL SEISMIC ACCELERATION.
·(7,8) PC STRENGTH, σ_{pc} = 100 AND 213 psi, RESPECTIVELLY.





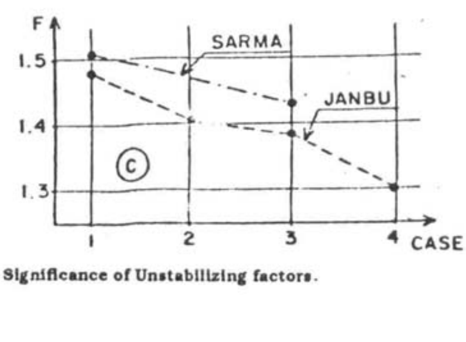


Figure 9: Quest of Cause effect - Significance of Unstabilizing factors.

immediately following query, "if it tends to fail, what changes of parameters, phenomena, and analysis would automatically tend to ensue"?

Firstly, for the same critical surface the F values were computed assuming compaction precompressions σp - 7 kg/cm^2 and 15 kg/cm^2 (hypothetical): if the higher σp values prove significant, one could induce manufacturers of vibratory rollers to develop, for experimentation, a roller of appreciably higher weight and impact stresses. Beyond the σp values two conditions may be used, one corresponding to the NC condition, which prevails during increasing height and stresses. Meanwhile for an eventual tendency to sliding after the stable end-of-construction condition has been proven, one should ponder on the validity of applying the PC strength equation, in as far as the very movement of the sliding mass should cause a decrease of vertical stress on the slip surface. The degree to which the NC and PC strength equations differ, favouring the post-construction stability compared with that of the rising fill, depends on the OCR, that is, on the amount of stress release, and therefore requires postulations of the changes of geometry to be produced by the slip.

The discussion is referred to Figure 8 and the accompanying Table I. Firstly, comparison of steps (A), (B) and (D), shows that for the failing surface 1 (NC condition) σp values of 7 and 15 kg/cm^2 would raise F from 0.98 $\rightarrow$ 1.05 $\rightarrow$ 1.68, an increase inviting investment in increased compaction stressing. A post-construction slip dropping by 10 m, accompanied by toe accumulation of equivalent volume (wedge) presumed to have degraded from PC to NC strength, would raise F from 1.05, 1.47 steps (B) to (C) this significant increase hints at a self-stabilizing tendency as soon as sliding would begin.

The upper stretches of the slide scar 1 were analysed, as per DETAIL (M), because of a recognized facility for falls, raveling and rolling of rock back to slopes less negative or subvertical. Despite using the unfavourable NC strength equation pertaining to increasing loading, the sliding analysis still evince a minor favourable ΔF for the wedges back of the slide scar.

DETAIL N, step (F), shows another analysis of an upper slip surface, 2, where the slide scar has outer slopes steeper than the 55° that led to NC failure. For these runs the PC condition of σp = 7 kg/cm^2 was used assuming the upper embankment built compacted to outer slopes 1(a), 1(b), 1(c), 1(d) (the latter coincident with the slide surface 1). The F values of 2.18, 2.14, 1.86, 1.75 confirm that compaction precompression precludes failure of shallow surfaces even in very steep slopes. Increased F values 0.98 $\rightarrow$ 1.03 $\rightarrow$ 1.07 $\rightarrow$ 1.10 result for deeper slip surfaces 1 $\rightarrow$ 2 $\rightarrow$ 4 $\rightarrow$ 5 even for pessimistic NC strengths (steps A, G). For the same surfaces corresponding values are F = 1.05 $\rightarrow$ 1.19 $\rightarrow$ 1.15 $\rightarrow$ 1.14 for assumed σp = 7 kg/cm^2 followed by NC strength for $\sigma v > \sigma p$ (steps B, H), and comparatively F = 1.68 $\rightarrow$ 1.41 $\rightarrow$ 1.45 $\rightarrow$ 1.52 assuming σp = 15 kg/cm^2 (σv does not exceed σp).

In most analyses hitherto the rockfill was conservatively assumed to crush and precompress under compaction, and not by overlying weight. It appears valid to consider NC conditions, applicable for $\sigma v > \sigma p$ as a lower bound while the fill is rising. However, for post-construction failure the material underlying the slide scar should develop PC strengths corresponding to σp and σpv (overburden stresses). The extreme hypothesis would be of "total removal" (evaporation) of the entire volume above slip surface 1, so that the stresses (driving and resistance-generating) are taken from the slices between surface 1 and the underlying surface under analysis (3, 4 or 5) and the PC strength available along these surfaces is taken for OCR conditions (Figure 8).

The differences of F results (Table I) under the two hypothesis are so flagrant (limit cases) that they call for careful consideration. But firstly a proviso must be emphasized regarding the computations summarized in sensitive comparative analysis one must choose between investigating changes of conditions on the same surface (erstwhile critical surface) vs. comparisons for various different surfaces, each one critical for its condition; in the interest of better exposing concepts, the first approach was preferred. The major benefit from the hypothetical high σp of 15 kg/cm^2 is obvious, and need not be pressed. The principal point, in gist, is that if there is a start of failure, unless there is a very "brittle" stress-strain behaviour, the tendency to equilibrium should be self-triggered. In conventional stability analysis, a hypothetical slip surface right under the real slide surface, would show an F value very slightly higher than the $\approx$ 1.0 value attributed to the sliding condition. If such were the reality in nature, a certain slide scar (of a significant mass removed) would be progressively subjected to further sub-parallel sliding. Nature shows that most often the surface right below a slide is quite stable, for many a year and decade: the inference is that the PC guarantees such stability, and only in materials that gradually lose the σp benefits by swelling, do slide surfaces themselves determine locations more chronically unstable than other virgin areas of the same original natural slope.

PURSUING THE SIGNIFICANCE OF F VALUES OF A CONVENTIONAL 1.0V:1.3H ROCKFILL SLOPE

Additional perspective on the significance of F values in rockfill slopes is obtained from other series of analysis under the following hypothesis:

1. For the basic critical surface adopted, recognizing that critical shearing and tensile conditions start at the

top, an extreme hypothesis was queried of a crack (or "totally lubricated" surface) progressing deeper, and attendant changes of F values computed, both for the NC and the PC ($\sigma p = 7\ kg/cm^2$) strengths. The complete stress release in the cracked stretch throws stress increments onto the remainder of the slip surface. The results, in Figure 9(a), show that the virtual crack would have to reach preposterous proportions (70 to 80 m deep) for the F values to drop to 1.0.

2. Again, for the same critical surface, assuming that tendency to failure be aggravated by soft loading on the crest, Figure 9(b) shows the computed decreases of F values with increased height of "soft load fill" (i.e. fill that does not develop any resistances within itself) on top. These runs were most conveniently based on Sarma's method, simultaneously giving the "critical horizontal accelerations" for instability. Correspondingly, the vertical (nominal) accelerations related to the vertical soft load applied could be obtained by direct ratios of applied forces to the weight of the critical sliding masses. Table 9(b) thus combined critical horizontal (Sarma) and vertical (nominal) accelerations of eventual seismic action for unstabilizing. Although the analysis used only the NC condition, realistic for the increased vertical loading hypothesis but pessimistic for the horizontal unloading hypothesis (Sarma), the results eloquently prove that even a modest F value (say of about 1.3) would yet require too high a physical loading for unstabilizing. For instance, for the original critical surface, a height of soft load fill of density $2.2\ t/m^3$ (137 pcf) would have to extend higher than 100 m to lower F to about 1.1.

For another manner of assessing the influence of heavier loading, comparative runs used increased rockfill densities of 4.4 and $6.6\ t/m^3$, simulating increased constant vertical accelerations. Also, further isolated comparative runs were made (a) applying the vertical soft load only on top of the sliding mass, (b) checking the critical horizontal acceleration (Sarma) for cases of compacted rockfills with PC envelopes. All these results are tabulated in Figure 9(b), series b1, cases 5 to 8, and confirm very favourable "resilience" exhibited by rockfill slopes to unstabilizing factors.

3. Finally, for yet another assessment of the physical capacity to unstabilize the basic critical slip surface, comparative runs investigated the effect of eventual removal of material from the toe of the sliding mass (NC strength). Both Janbu and Sarma runs were used, Figure 9(c): for convenience the wedges removed were chosen pertaining to the original analysis (case 2, Figure 3). Surprisingly big wedges must be removed to reduce F from 1.5 to 1.3.

These three sets of analyses surely confirm the inherent difficult to unstabilize a rockfill slope of conventionally denominated angle of repose, even under NC conditions. Compacted rockfill under PC conditions should be very much better.

CONCEPTUAL JUSTIFICATIONS FOR THE CONSPICUOUS DIFFERENCE BETWEEN SLOPES IN SANDS AND IN ROCKFILLS

Conventional soil mechanics has driven so deep the concepts regarding slopes of sand stockpiles, angles of repose, the slope stability factor $F = \tan\phi/\tan i$, the influence of density, and the generally controlling loose condition at the slope, that understandably the faces difficulties and momentous responsibility in advancing these theses.

Some light, and hint of promising rationalization, is extracted from the data on end-dumped rockpiles (loose, NC strength conditions) reported by de Mello[8], differentiating between the surveyed stable slopes during filling and during bottom excavation; but it must be aided by intuitive analysis. The data published are transformed in Figure 10(a) into rough histograms and frequency distributions. The concepts should be extendable to compacted rockfills, PC conditions, guaranteeing much higher F values, since in the field data used there are no benefits from compaction crushing hysteresis.

In any material exhibiting a dispersion of strength parameters around the mean, if we temporarily assume constancy of the stressing towards failure, the classic definition of factor of safety F should be differentiated from two other factors FG = Factor of Guarantee, and FI = Factor of Insurance. The three definitions were simply defined as (e = dispersion):

$$F = \frac{strength \pm e}{stress}\ ;\ FG = \frac{strength + e1}{stress}\ ;\ and$$

$$FI = \frac{strength - e2}{stress}$$

and obviously a given structure is increasingly safer for the same numerical value of FI $\rightarrow$ F $\rightarrow$ FG. The typical F condition occurs when there is no possibility of anticipating to which side of the average strength the uncertainty e will develop: FG prevails when a "protesting condition" guarantees that the strength can only prove higher than a demonstrated value; finally FI would apply when the strength applicable to the problem can only be worse than a value determined. The differences between FG, F, and FI values in a specific problem depend on the spread of the frequency distribution curve, and on the manner in which the pretesting condition truncates it, at the lower end for FG conditions, or at an upper limit for FI situations.

In Figure 10(b) a plausible intermediate distribution curve, GENERAL, G, for the given rockfill has been fitted. The repose slope in sands, gravels, and rockfills is determined by the stability of individual

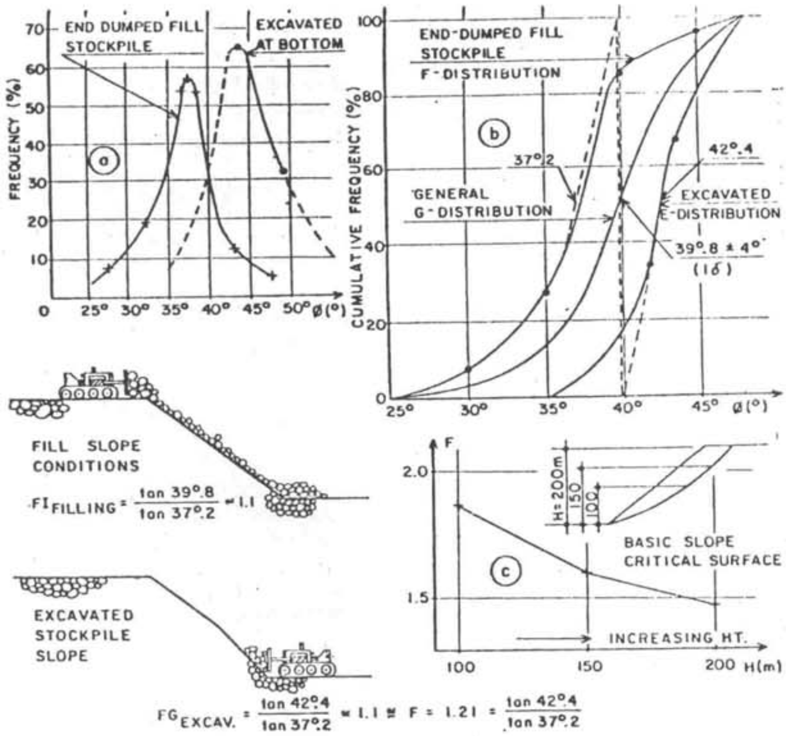


FIGURE 10: Stable Slopes on Rockfill Stockpiles and Conceptual Analysis of Factors of Safety

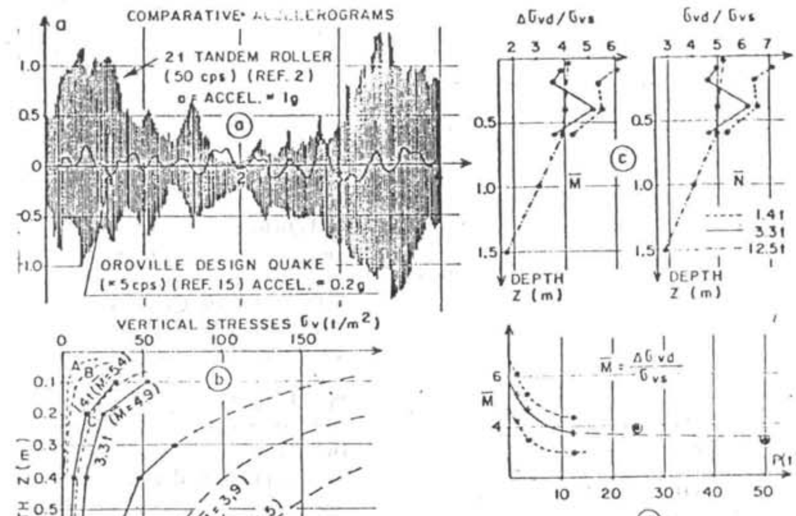


FIGURE 11: Comparative Dynamic Stresses, Roller Versus Seismic

blocks at the surface, controlled by interlocking at zero applied stress: thus the flatter slope during stockpiling is controlled by the least interlockable rock fragment, pushed over the brink and rolling, having to stop, a situation recalling the difference between "static" and "dynamic" friction. It is thus representative of the worse conditions within the rockfill dispersions; small rock volumes of much greater stability and localized slope could occur, but cannot about proportionally so as to represent the slope; if they are too stable for the slope angle they cannot roll and stop midway to form part of it, but will be surrounded by median stable rocks and therefore be buried below. In short, we may consider the G-distribution truncated excluding the values higher than the average, yielding the FILL, F, distribution. Moreover, we infer that in such an end-dumped rockfill pile increasing in height the slopes imply FI conditions from the overall end-dumped rockfill properties. The slope is, however, quite stable on average; end-dumped rockfill slopes are not subject to continuous raveling or surface sliding deformations. Roughly representative values for the factors of safety are given in Figure 10 for the example used.

Recapitulating, at least three factors have imposed the flatter slope of rockfill material end-dumped: the dynamic vs. static friction condition, the loose non-imbricated condition, and the inference that blocks of high — ϕ well — imbricated behaviour will inevitably occupy a position somewhat below the surface. Therefore, the internal critical surface is confirmed, for eventual mass sliding in the rising NC fill. Thereby the concept reports to the change of F as the fill rises, a steady deterioration being proved [Figure 10(c)], forcibly demonstrating that on all counts an end-dumped rockfill constructed to a certain height has been pretested to the height. Any unsatisfactorily stable rocks and/or slide volumes would have been removed "by natural selection", and all the material that remained was better than the lower bound average value proven by the mass sliding stability reached at the top. Thus, for conditions after end-of-construction, the slope stability can be concluded to be proven into an FG condition on the initially FI values. Moreover, for any unstabilization caused by excavation of the stock-pile at the bottom, favoured FG conditions prevail : the values have to be equal to, or better than, some pretested value. During the bottom excavation, small representative volumes of surface rocks are held stable by the bigger blocks well imbricated ("static friction"), thus guaranteeing the stability of a much steeper slope.

For both the situations conceived, of FI rising-fill, and FG bottom-excavated stockpile, the truncation (rejection criterion) of the G-distribution has been assumed at the median ϕ, giving the F and E-distributions. Under these assumptions (or any analogous one) one derives the comparative FI, F and FG values listed in Figure 10, under the simple assumption of linear cohesionless envelope, and $F = \tan\phi 1/\tan\phi 2$.

Three corollaries arise from these discussions on such a generic case represented by clean angular sound rockfill and frequency-distribution curves. Firstly, for any moderately sloping frequency distribution curves of ϕ values at low stress, a given external "angle of repose" slope has a stable factor of safety. Secondly, since in sands a much lower numerical F value can be accepted than in clays (de Mello,[9]), because of the much smaller dispersions (no dispersion of pore pressure u), the same reasoning reinforces acceptance of very low F values as unquestionably satisfactory in rockfills. Thirdly, the analysis of accepted dogmas on repose slopes in uniform sands fits easily as an idealized material of G-distribution curve absolutely vertical, and without any hysteresis of crushing of grains (near surface). All three distribution curves, F, G, E will coincide, and FI, F, FG = 1.00. Inevitably therefore the angle of repose condition tends to F = 1.00, and any grain on the slope is on the verge of slipping or rolling. These facts fit into the general theory, but in no way suggest that a "repose" slope of rockfill has F = 1.00.

PRELIMINARY IMPLICATIONS ON SEISMIC PROBLEMS

Immense efforts have been applied to cyclic testing of large-size laboratory specimens of rocks, and to finite element analysis of the predictable permanent displacements and yield accelerations or rockfill dam slopes under hypothetical critical earthquake. The authors are unable to confirm to what extent the test results and subsequent analysis embody the benefits of precompressions by crushing. However, the engineering philosophy postulated by de Mello[9] stands most applicable in the highly unpredictable and especially uncheckable field of seismic forecasts and behaviours, to the effect that one should prefer to predict and guarantee what will not happen, rather than attempt to predict what might be likely to happen (cyclic compressibilities and yields) in the compacted rockfill dam. In short, if at all possible, the engineer should aim at achieving a compaction treatment such that the rockfill might be pretreated and pretested to behave satisfactorily under the eventual design earthquake.

Since vibratory-impact compaction is used in the construction, the first step comprises checking to what extent the dynamic stresses applied during compaction might envelop the predictable seismic stresses in the significant zones of the rockfill. Such data as shown in Figure 11(a) suggest the possibility of achieving the aim regarding impact stresses, accentuated by much higher accelerations: the problem lies in pre-compensating for high static overburden stresses to which the lift is subjected when the cyclic seismic stress superposes. Recognizing that in many a case the aim may not be fulfilled by commonly used equipment, the second step involves assessing how to adjust equipment, materials, and construction specifications, to meet the intent. It may often prove much more effective, technically and economically, to promote the desired adjustments in the manufacture of vibratory compaction equipment, and to adjust material and construction specifications, for the desired level of stressing of the rockfill, so as to dictate its behaviour, than to accept passively the present routines that had not considered seismic problems, and to spend most efforts on attempting to predict the behaviour of the material under debatable and uncheckable premises. When many millions of cubic metres of compacted soils were involved, improved rollers were developed for the purpose: an analogous situation now exists for the much increased use of compacted rockfills.

The present discussion merely intends to open vistas to the approach presumed promising; the fact is that many research and practical developments should be detailed (e.g. Marsal,[16], Schwab et.al.,[19], etc.), but would require a full additional paper.

Broms and Forssblad[2] list as principal factors affecting vibratory compaction efficiency, the: (a) resonance frequency of the compactor-fill system; (b) number of cycles of loading; (c) shear resistance during vibration; (d) magnitude of dynamic stresses transmitted to the lift. For rockfill, factors (a) and (c) that control rearrangement of grains lose importance in comparison with factors directly related to stresses crushing point contacts, which one accepts as related to (e) measurable average stresses (function of weight and impact of roller) and (f) the ratio of impacting weight to weights of impacted stones, and especially (g) grain shapes and size distribution. The increasing rigidity of lift with increasing passes has been finally established as the really rational measure of improved compaction (cf. Schwab et.al.,[19]): in as far as this increased rigidity is due to contact stresses, it inevitably reflects decreased deformability (static and cyclic) and increased PC strength and slope stability.

Data for attempts at forecasting are very scant but consistent enough to offer positive promise. Figure 11(b) reproduces vertical stresses transmitted into the lift by loads at the surface (a) statically, σvs, by calculations for a Boussinesq medium, and (b) dynamically, σvd, as measured for 1.4, 3.3 and 12.5 ton rollers. Possible curves for σvd, for eventual 25 and 50 ton rollers are also plotted as per Figure 11(c). In the latter graphs simple ratios of dynamic to static stresses from Figure 11(b) at different depths are used for extrapolations, plausible though crude. An advantage of using shallower lifts derives from the higher stresses transmitted [Figure 11(d)] but one should recall that thinner lifts represent smaller stone, and the effects will differ considerably if quarry run finer rock is compared with crushed sizes, relatively uniform and stronger.

Figure 12(a) reproduces as an example the maximum seismic stresses Δσvd/σvs computed by FEM analyses for the Las Piedras Dam (Parras and Cervantes, 17) for a M = 7 earthquake; the incremental vertical stresses reached about 15% of the static overburden stresses. Figure 12(b) shows that as

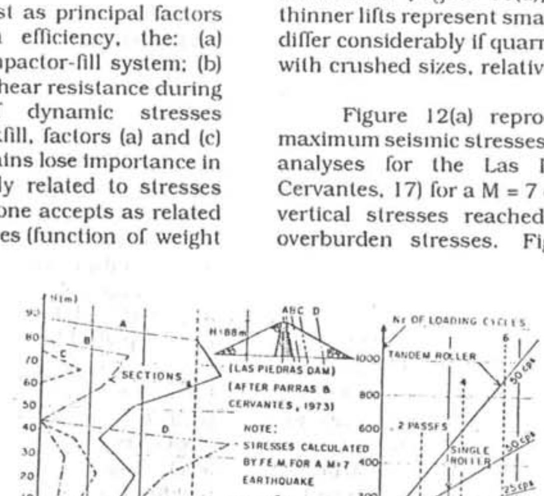


FIGURE 12: Possible Rockfill Compaction Treatment for Seisms

regards number of loading cycles the rollers are highly superabundant in comparison with an earthquake (e.g. Oroville case). Using the 15% increment [Figure 12(a)] and a vibratory roller frequency up to twice the present routine equipment, Figure 12(c) shows approximately the maximum layer thicknesses to be used so that the roller dynamic stresses will precompress the lifts to crushing equivalent to $\sigma p = 115\% \sigma p\gamma z$. The indications are obvious, but the orders of magnitude must be emphasized. A 12.5 ton roller may roughly meet requirements for 50 m burden, under earthquake, with lift thicknesses of 0.4 to 0.5 m. One must recall, however, that for slope stability purposes the overburden height in question is that on the eventual sling surface (Figure 3). One final important point of optimization concerns the rockfill particle sizes, quality and shape. Marse[15] suggests an index expression, Figure 12(d), for the average contact force as a function of particle sizes and average applied stress. Assuming a concern that by exceeding some intact force (e.g. herein taken as 100 kg) the rock particle would offer breakage, Figure 12(d) indicates the maximum particle dimensions ($1/n_s$) so as to avoid such a problem, thus imposing increasingly smaller sizes under heavier rollers. The concept requires region recognizing that material from a given quarry and crushes inexorably exhibits higher intrinsic resistances ("by natural selection") as bigger sizes possessing weaker more distant planes are crushed to smaller ones, with only stronger weakness planes surviving.

The topic can only be opened for research and development. An ideal material and grain size distribution should be one that would suffer significant crushing of infinitesimal angular contacts, so as to reach high σp stresses on a minimally incremented contact area, of really high resistance to further crushing.

The fact is that even with clean angular rockfill of qualities far from ideal, the crushing precompression already improves very much both the static and the seismic slope stability and behaviour. Any concentrated effort on research and development along the lines suggested should offer high benefit/cost ratios to dam design and construction.

REFERENCES

(1) Barton, N. and Kjaernsli, B., "Shear Strength of Rockfill" ASCE, Vol.107 (GT7), Proc. Paper 16374, July 1981 — p. 873-891.

(2) Broms, B. and Forssblad, L., "Vibratory Compaction of Cohesionless Soils" — Proc. of Spec. Sess. 2, 7th ICSMFE, Mexico 1969 — p.101-118.

(3) Charles, J.A. and Soares, M.M., "Stability of Compacted Rockfill Slopes", Geotechnique 34, n.1, 1984 — p. 61-70.

(4) Charles, J.A. and Watts, K.S., "The Influence of Confining Pressure on the Shear Strength of Compacted Rockfill" Geotechnique, Vol.30, N.4, 1980, p. 353-367.

(5) Costa Filho, L.M. and Thomaz, J.E.S., "Stability Analysis of Slopes in Soil with Non-linear Strength Envelopes Using Non-circular Slip Surface", IV International Landslide Symposium, Toronto 1984, Vol. II, p. 393-397.

(6) de Mello, V.F.B., "Behaviour of two Big Rockfill Dams, and Design Aims", International Conference Case History in Geot. Eng., Rolla (Missouri) 1984, Vol. 2, p. 923-942.

(7) de Mello, V.F.B., "Comparative Behaviours of Similar Compacted Earth-rock Dams in Basalt Geology in Brazil", Symposium on Problems and Practice of Dam Engineering, Bangkok 1980, Vol. I, p. 61-79.

(8) de Mello, V.F.B., "Some Problems and Revisions Regarding Slope Stability Assessment in Embankment Dams", Symposium on Problems and Practice of Dam Engineering, Bangkok, 1980, Vol. I, p. 81-89.

(9) de Mello, V.F.B., "17th Rankine Lecture: Reflections on Design Decisions of Practical Significance to Embankment Dams" — Geotechnique 27, No.3, 1977, p.281-355.

(10) de Mello, V.F.B., "Concrete Dam Foundations : An Open Case of Geomechanical Interaction, Structure-Foundation and Theory-Practice" — 4th Australia-New Zealand Conference on Geomechanics, Perth 1984, Opening Lecture.

(11) Forssblad, L., "Investigations of Soil Compaction by Vibration", Acta Polytechnica CE No.34, Stockholm 1965.

(12) Fredlund, D.G., "Analytical Methods for Slope Stability Analysis", IV International Symposium on Landslides, Vol.I, p. 229-250.

(13) Gazetas, G. — "Vertical Oscillation of Earth and Rockfill Dams, Analysis and Field Observation", Soils and Foundations, Vol. 21, No.4, December 1981.

(14) Janbu, N., "Slopes and Excavations", General Report by Morgenstern-Blight-Janbu-Resendiz, IX ICSMFE, Tokyo 1977, Vol. 2, p. 549.

(15) Leps, T.M., "Review of Shearing Strength of Rockfill", ASCE Proc. SM4, July, 1970, p. 1159-1170.

(16) Marsal, R. "Mechanical Properties of Rockfill" Embankment Dam Engineering, Casagrande Volume, 1973, Wiley, p. 109.

(17) Oroville Earthquake Investigations, State of California Department of Water Resources, February 1979.

(18) Parras, Y. and Cervantes, R., "Analysis de Presas de Tierra Y Enrocamiento Sometidas a Temblores", Unam 310, Enero 1973.

(19) Sarma, S.K., "Stability Analysis of Embankments and Slopes", Geotechnique 23, No.3, 1973, p. 423-433.

(20) Schwab, Pregl and Kieres, "Compaction Control with the Compactometer", VIII ECSMFE, Helsinki, 1983, Vol. I, p. 73.

(21) Sultan, H.A. and Seed, H.B., "Stability of Sloping Core Earth Dams", ASCE Proc. SM4, July 1967, p. 45-83.

(22) Veiga Pinto, A.A., "Prediction of the Structural Behaviour of Rockfill Dams", Thesis, LNEC, Lisbon, 1983.