78.1

THIRTEENTH CONGRESS ON LARGE DAMS, CIGB/ICOLD,
New Delhi, 1979, Vol. V, pp.350-352. Q.48.

42 - Victor F. B. DE MELLO (Brazil)

Mr. Chairman, fellow Delegates. Of the many points very ably summarized by the General Reporter, presented in the papers, and under debate at this Session, I restrict my comments to three singled out in the General Report.

1. FABRIC FILTER-DRAINS

The General Reporter's question would aim at problems of possible long term biodegradation : first serious concern should, however, focus on the interface problem of difficulty of snug contact fabric filter and compacted soil slopes, as well as incompatibility of stress-strain-strength behavior between soil and fabric, as a result of which there is frequent localized wrinkling and creasing, and fairly rapid erosion of unconfined underlying soil, and localized clogging at key positions.

In some fabrics, tests have shown interface Φ values of the order of 8 to 11° with compacted clay-silts of $25° < \Phi < 30°$, and the strains to such ultimate Φ values are of the order of 5 to 10 times the soils' typical 2 to 8 % to failure. The use of such fabrics as a filter under rip-rap, in substitution for the well-proven preference for interlocked well-graded quarry-run fine rock spread from interface to outerface rip-rap, has resulted in many an embarrassing failure. Even in soils in which one would rely on compacted cohesion, it must be recalled that one of the functions of the granular transition is to distribute evenly the rip-rap overburden pressure in minimum value to exclude free swelling : with fabric filters the interface pressure under the filter is extremely varying because of point contacts of angular rock and of spanning by the tensile resistant fiber, and tends to be too low, as generally rock cover is minimized over fabrics. Inevitable surface imperfections in construction must also be considered. Numerous cases may be cited in which after initial wrinkling of the fabric due to downward strain, upon first drawdowns interface erosions and local accumulations of silty soil occur, and upon a subsequent rapid drawdown there have been push-out and slip failures along the underside of the fabric as if it were insufficiently pervious.

2. USEFULNESS OF FINITE ELEMENT (FEM) ANALYSES

Nobody can deny the usefulness of FEM for analysing the more complex problems of stress and strain distributions, and especially for evaluation of the influence of a given parameter. It can be affirmed, however, that for embankment dams : a) many an answer can be extracted firstly from analytical solutions, such as by Davis and Poulos, assuming homogeneous embankments first of one material, then of the other material, the design being thereupon implemented by the all-important decision to adjust the zones and materials so as to be as nearly homogeneous as possible; b) the FEM analyses would only be a subsequent step for situations in which specific heterogeneities are still unavoidable; c) the problems lie in the determination and selection of appropriate parameters, and later, after the computations, in adjustment of results to representing the prototype, and finally in deciding on levels of acceptance of the predicted behaviors.

Laboratory compacted specimens (design phase) aim at identical physical properties but achieve them by confined compression within metal moulds, whereas in the field the lift undergoes local compression and shear under the difference of roller pressure along the strip and zero pressure adjacent, and generally the limiting roller pressure is conditioned by the corresponding bearing capacity condition. Moreover, if one considers " undisturbed " samples taken from compacted fill, it has been repeatedly demonstrated and discussed that the conventional soil mechanics tests on " undisturbed specimens " are far from reflecting stress-strain-time paths representative of " intact " soil elements (Skempton and Sowa 1963, Ladd et al. 1965, Seed et al. 1965, Poulos and Davis, 1967). Finally, in a compacted embankment the soil element starts its overburden loading trajectory from a condition of high lateral stress built-in by compaction (cf. my Rankine Lecture, Geotechnique 1977) and the stored energy of compaction in both clays and angular rockfills reflects in a " preconsolidation pressure ". The urgent need is to recognize the failings of laboratory and in situ tests as routinely conceived, and to adjust special testing techniques to the needs of computational techniques that are decades ahead.

3. CRACKING OF CORES AND CORE/GRAVITY-WALL INTERFACES

This is yet another all-important topic in which one must urgently rid the profession of some misnomers and much-quoted qualitative illusions. Obviously the first emphasis is on the significant difference between tight shear-plane " cracking " under high normal stresses, and tensile cracking : only the latter is of concern, and it is easy to see why it can only occur at the top of embankments and interfaces, although much influenced by what was executed directly below. One should favour high compaction (even if somewhat dryer) and low compressibility in lower reaches. At the top, in job conditions one cannot think of relying on tensile resistances, which belong to " weakest-link theory " extreme-value statistics and will be zero at some unpredictable plane : moreover, there is no possible direct correlation between Plasticity Index PI and magnitude of " plastic deformation " to failure of compacted clays at low confining stresses, nor can there be any quantifiable reliance on minute differences of tensile strainability to failure as affected by materials and compaction specifications (under heavy modern equipment that is the conditioning factor); brittleness sets in by thixotropy, etc. and the rate of change of reservoir loading may be the worst culprit in cases where, for instance, first filling reaches 60 m in about 20 days (e.g. the Salto Osorio and Salto Santiago earth-rock dams in Brazil).

The important design principles postulated in my Rankine Lecture (1977) would favour a) avoiding any tendency to creation of tension zones (in analyses of " homogeneous " material compressing); b) construction pore pressures about as high as final reservoir flownet ones, so as to minimize change of conditions on reservoir filling. Regarding the core/concrete-gravity interface it was emphasized that natural movements at the top render the core contact on the upstream face unreliable. Finally, the steep transverse face contact with minimized clay-concrete strength adherence (but maximized impervious snugness) should be preferred, since to the extent to which there is " plastification " (i.e. shear failure) there will be no opportunity for hang-up and any local tension crack at the single weakest plane, to absorb the tensile strain from the differential settlement. A 1 : 100 distortion (specific differential settlement) causes an extension of 5×10^{-5} of the hypotenuse, corresponding to a single crack 1 mm wide in a crest length of 20 m : distortions of 1 : 100 have been accepted as estimated limits in some dams, and are somewhat akin to those discussed as damage thresholds of brick wall-panel buildings.