Deformation of Earth/Rockfill Dams

Déformations des Barrages en Terre ou en Enrochements

R.J.Marsal Prof., Universidad Nacional Autónoma de México, Mexico

The Organiser of the Session, Prof Raúl J Marsal (Mexico) made a short introduction. He first thanked the Organising Committee for its invitation to promote the discussion on deformation of earth-rockfill dams. He then proffered the appologies of his Co-Organiser, Prof A W Bishop, who was unable to attend the conference for health reasons. The three members of the Panel were introduced: Jesús Alberro, Research Professor of the Institute of Engineering, National University of Mexico; Stanley D Wilson, Executive Vice-President of Shannon & Wilson, Inc, Seattle, USA; and Peter R Vaughan, Reader in Soil Mechanics, Imperial College, London, Great Britain. Prof Marsal also introduced the Secretary of the Session, Mr Toshio Fujii, Deputy General Manager, Construction Department, The Tokyo Electric Power Company, and Mr Raúl F Esquivel, Federal Electricity Commission of Mexico, who had acted as Assistant to the Organiser.

The programme of the session was arranged as follows:

Part 1. Short presentations by the panelists, followed by a discussion between them.

Intermission to receive notice of short contributions and questions from the floor.

Part 2. Short presentations from the floor on the topics nominated for discussion, and questions from the floor, together with open discussion between the panelists and the audience.

The topics for discussion were

1. Hydraulic fracturing
2. Plastified zones and progresive failure
3. The use of the Finite Element Method for design purposes.

Prof Marsal said that he would save time by not reading his own key-note paper on "Strain Computation in Earth-Rockfill Dams", which had been circulated in the session pre-print. The proceedings of Speciality Session 8 would be published and would contain the key-note paper, the three papers by the panelists, the minutes of the session, contributions from the floor and those presented in writing after the conference.

Part I PRESENTATIONS BY PANELISTS

The three members of the Panel made brief presentations of their papers.

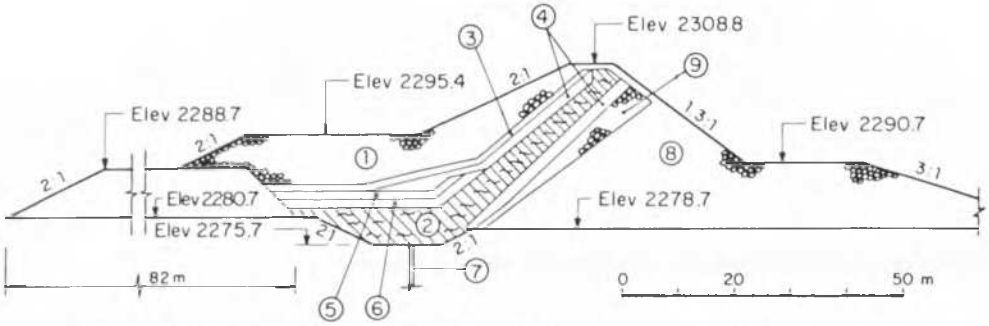
Prof Jesús Alberro discussed hydraulic fracturing and plastified zones within an embankment.

Hydraulic fracturing occurs, if any, during the first filling or the drawdown of the reservoir; it also occurs when a hole stabilized with mud is drilled in the core of a dam. In such cases, effective tensile stresses may develop and induce cracking. This phenomenon was illustrated with observations made at section 0 + 120 of Guadalupe Dam. Figs 1, 2 and 3 show, respectively: 1) the cross-section of the embankment; 2) values of total and effective principal stresses measured by pressure cells and piezometers located in the impervious zone, and 3) strains registered with linear extensometers, also installed in the clay core. Note that, during the drawdowns of 1971 and 1972, tensile principal stresses developed and that corresponding extensometers 5 and 6 showed extension strains, thus revealing the formation of a crack.

The stress changes in the embankment consequent upon the first filling were described. The equation frequently used,

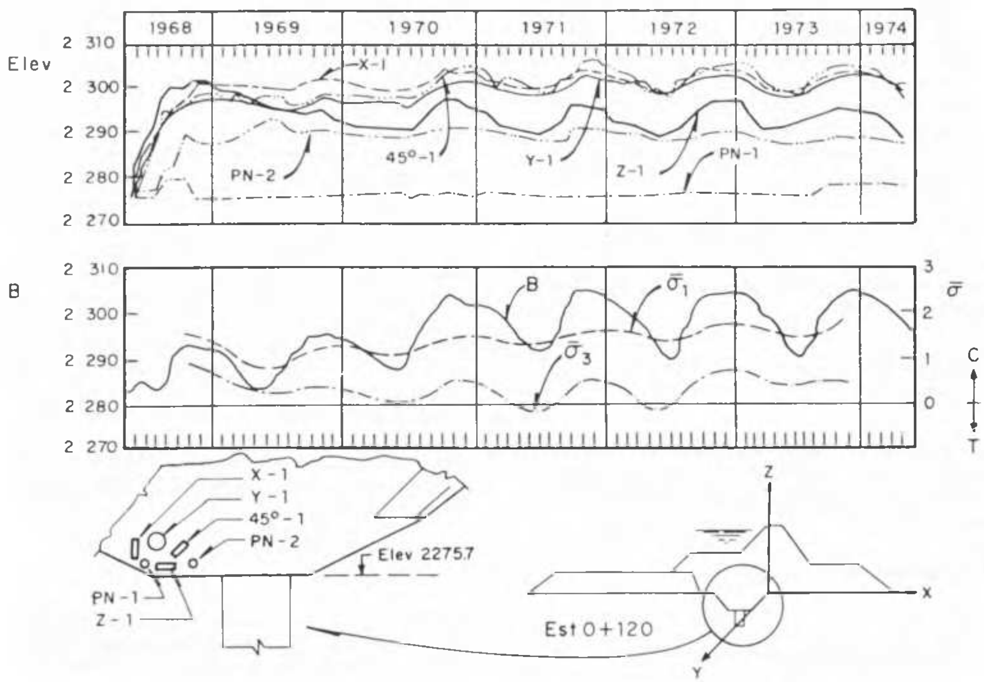
$$\sigma_z < \gamma_w h$$

(where σ_z is the total vertical stress at the end of construction, γ_w the unit weight of water and h the vertical distance between the given point and the pool level) although sufficient by itself, is not a necessary conditon. The computation of the state of effective stresses in a core during filling, taking into account seepage forces, bouyancy and interaction between zones, discloses the influence of three factors in the evaluation of hydraulic fracturing potential: a) the state of stress at the end of construction; 2) the geometry of the core, and 3) the anisotropy of the impervious zone from the viewpoint of permeability.



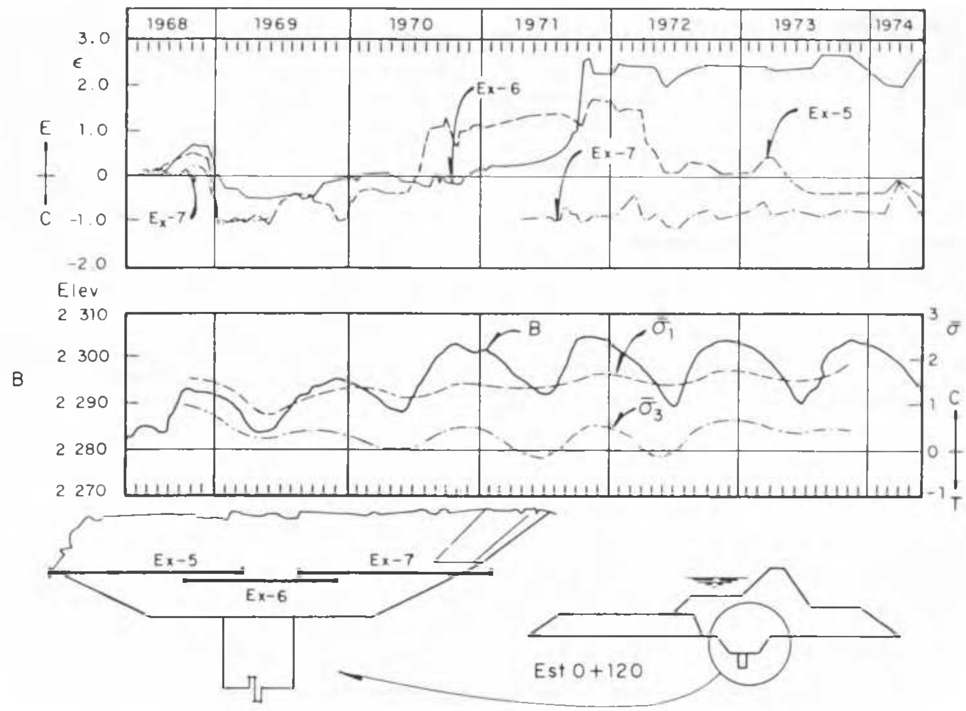
Elev Elevation, in m
1) Rockfill
2) Impervious core
3) Selected muck transition
4) Sand and gravel filter
5) Clean fine sand
6) Highly plastic clay, lightly compacted
7) Cut-off wall
8) Earthfill
9) Rock fines transition

Fig 1. Maximum cross-section of Guadalupe Dam



X,Y,Z,45° Pressure cells
PN-i Pneumatic piezometers
C Compression
Ex-i Extensometers
Elev Elevation, in m
B Pool level, in m
$\bar{\sigma}$ Effective principal stress, in kg/cm^2
$\bar{\sigma}_1$ Major principal stress
$\bar{\sigma}_3$ Minor principal stress
T Tension
Est Station, in m

Fig 2. Data recorded from pressure cells and piezometers 1 and 2, Sta 0 + 120, Guadalupe Dam



ϵ	Strain, in per cent	Elev	Elevation, in m	$\bar{\sigma}_3$	Minor principal stress
E	Extension	B	Pool level, in m	T	Tension
C	Compression	$\bar{\sigma}$	Effective principal stress, in kg/cm^2	Est	Station, in m
Ex-i	Extensometers	$\bar{\sigma}_1$	Major principal stress		

Fig 3. Data recorded at extensometers 5, 6 and 7, Sta 0 + 120, Guadalupe Dam

Consequently, to minimize the hazard of hydraulic fracturing it is recommended that: 1) the stress σ_z at the end of construction be increased, either by changing the geometry of the core or by raising the compressibility of the transition zones so that they will hang on the core; 2) lamination of the impervious zone be avoided; 3) the rate of filling be reduced in order to minimize the effect of transient seepage, and 4) the material selected for the core should not be susceptible to erosion and be protected by generous filter zones.

In conclusion, to evaluate the hydraulic fracturing potential a study of the stress states for different load conditions is necessary. This can be accomplished by means of the finite element method, the reliability of which depends upon the parameters used to duplicate the mechanical behavior of the soil.

The plastification or local shear failure of certain zones of an embankment was the second topic discussed by Alberro. Field measurements in the cases of El Infiernillo and La Angostura Dams, revealed that the above phenomenon is a fact and not an hypothesis based upon more or less dubious calculations. The prediction of plastified zones should be used as a guide for locating the potential slip surfaces within an embankment. It was also indicated that, even for a factor of safety greater than unity, the core may be affected by zones subjected to local shear failure, where settlement increases and shear strength diminishes. Furthermore, the continuity of these zones may have a significant influence upon the behavior of the dam.

Stanley D Wilson condensed his remarks about the influence of field measurements on the design of embankment dams as follows.

The internal movements that typically develop in embankment dams during the construction phase are illustrated in Fig 4. Both maximum settlement (vertical movement) and horizontal movement occur at mid-height with an approximately parabolic distribution. Along the profile of the dam, points on the two flanks move towards the center, and the shells bulge upstream and downstream. During first filling of the reservoir, Fig 5, the maximum movements, both vertical and horizontal, take place along the crest and the slopes of the embankment. The crest develops extension strains near the abutment slopes, and compressive strains in the center. It is during this period that transverse cracking is most likely to develop. In the upstream-downstream direction, the upstream shell settles and pulls away from the core, reducing the earth pressure against the upper part of the core which then may actually deflect upstream.

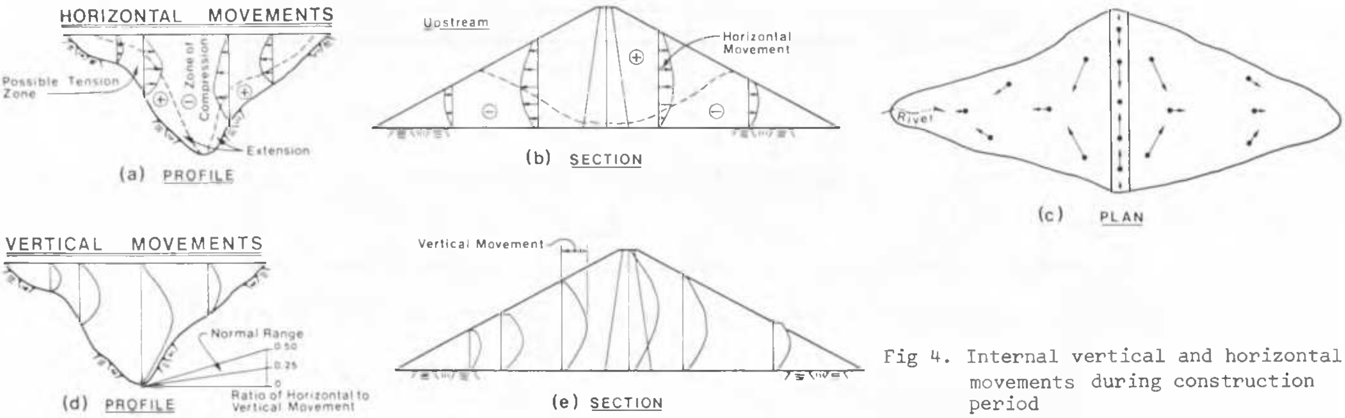


Fig 4. Internal vertical and horizontal movements during construction period

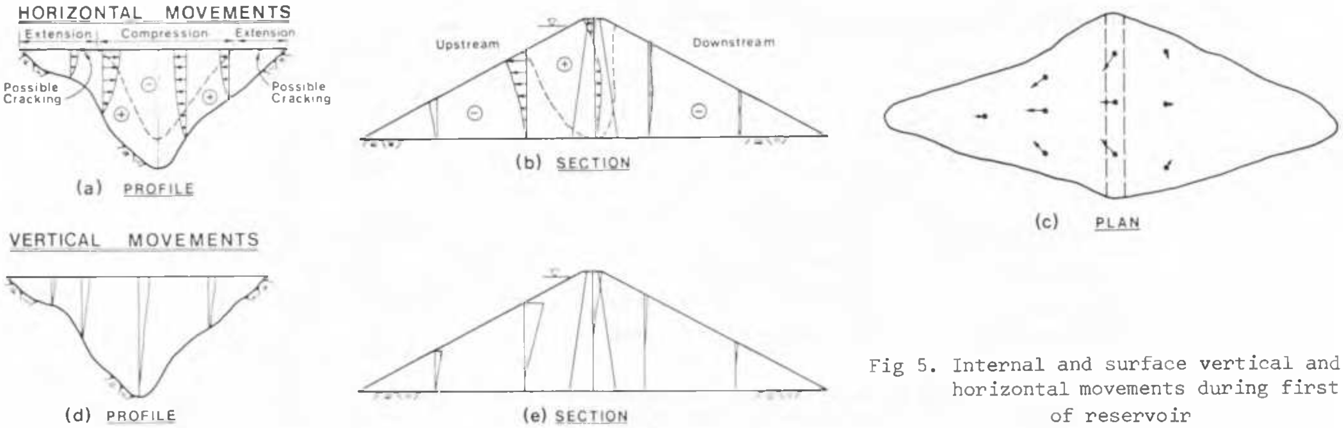


Fig 5. Internal and surface vertical and horizontal movements during first of reservoir

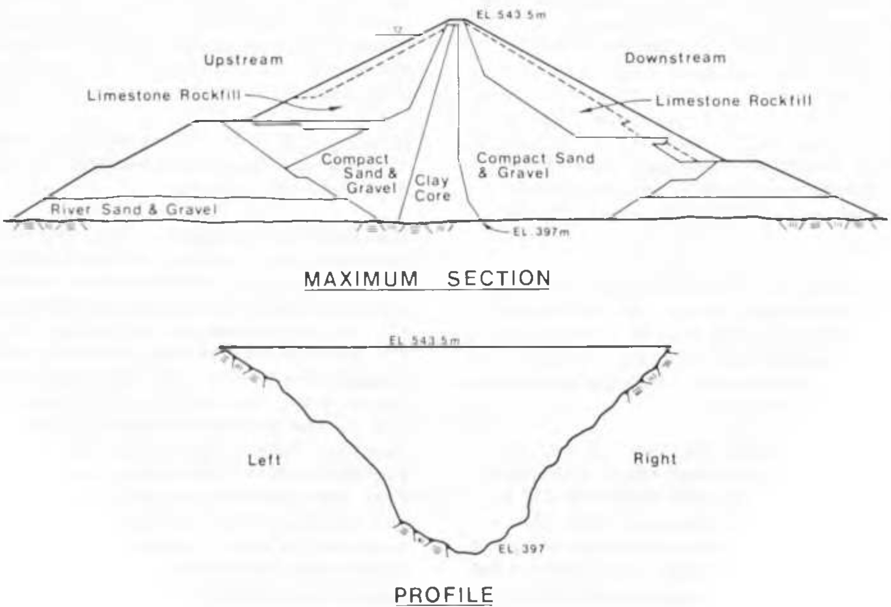


Fig 6. La Angostura Dam, Mexico

When the embankment is composed of many zones, incompatibility in compression between various zones may lead to unusually large differential movements between adjacent zones, as can be illustrated with field performance data from La Angostura Dam on the Grijalva River in Mexico, Fig 6. A vertical core of clay was supported by zoned shells of excellent sand and gravel flanked by highly compressible weathered limestone, compacted in layers. The comparative compressibilities of these materials are shown in Fig 7. The limestone rockfill compressed significantly under its own weight when flooded, although this is not indicated on Fig 7.

Fig 8 shows the distribution of settlements during construction along two horizontal planes and Fig 9 the settlement across the interface at the downstream face of the core. The average shear strain across a zone 2 m in width was about 15 per cent which means that the shear stress must have exceeded the shear strength, and thus the movement represents a state of plastic deformation.

During first filling the rockfill in the upstream shell settled, dragging down the thin vertical wedge of compacted sand and gravel as shown in Fig 10 and causing upstream and downstream movements as shown in Fig 11. The resulting surface movements are shown in Fig 12.

It is clear that compression of the various materials comprinsing a dam and its foundation may result in large differential movements between adjacent zones.

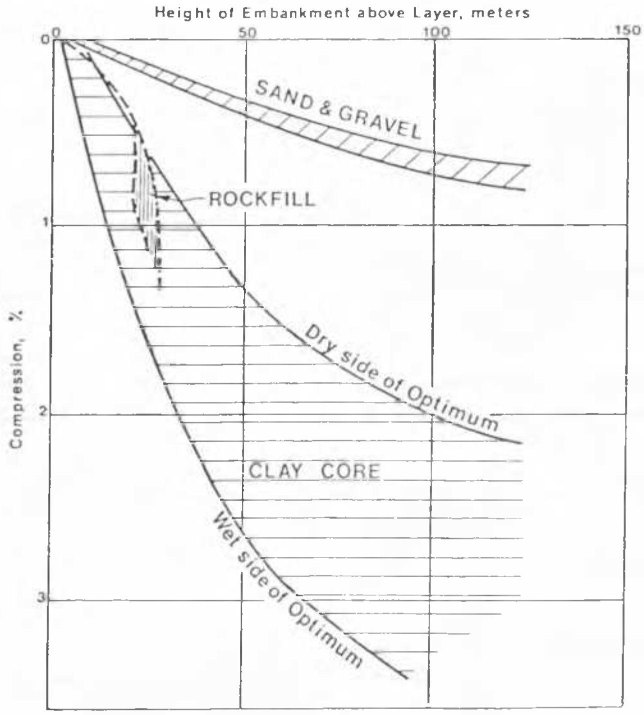


Fig 7. La Angostura Dam, Mexico. Compression of materials during construction

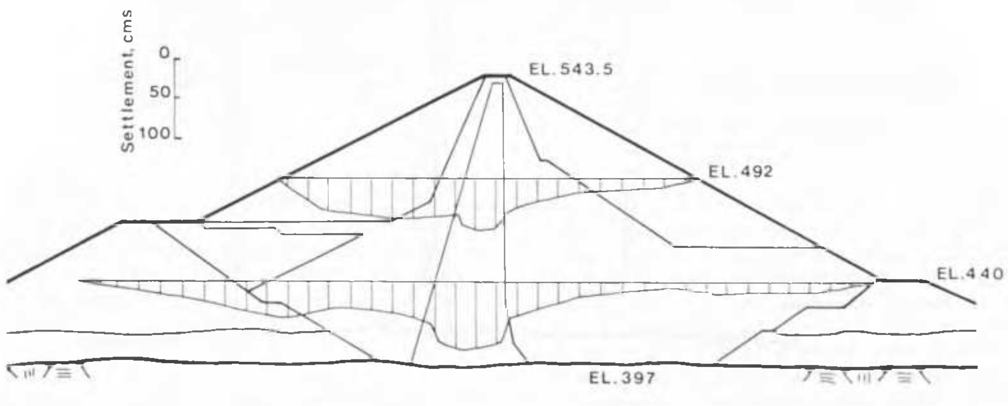


Fig 8. La Angostura Dam, Mexico. Settlement during construction of main section

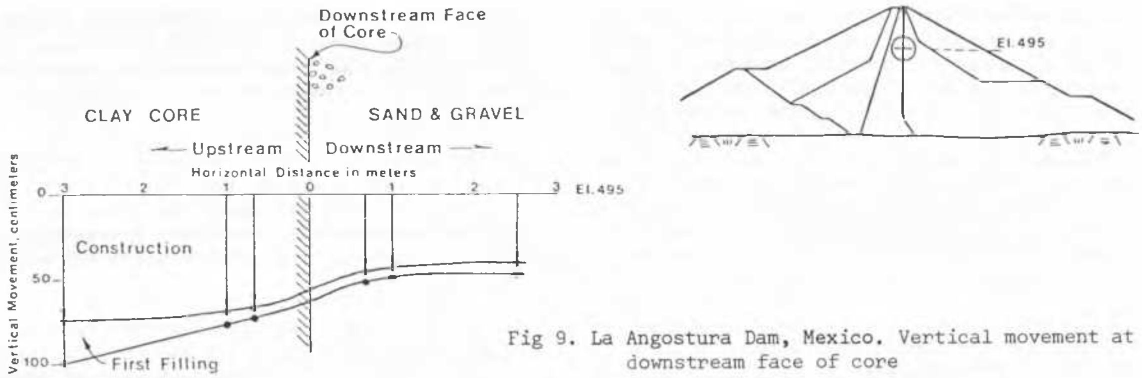


Fig 9. La Angostura Dam, Mexico. Vertical movement at downstream face of core

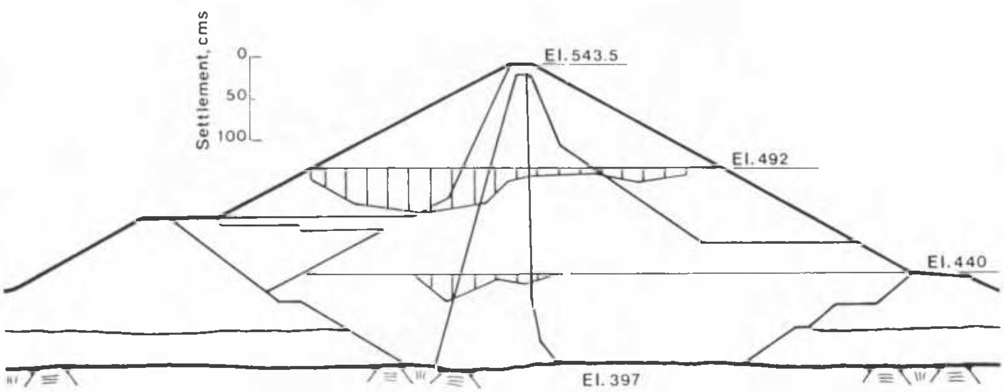


Fig 10. La Angostura Dam, Mexico. Settlement during first filling main section

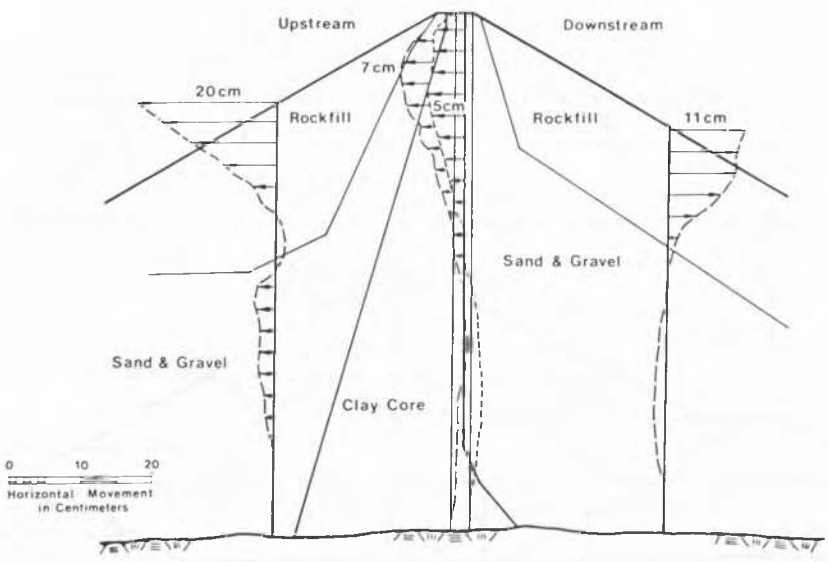


Fig 11. La Angostura Dam, Mexico. Upstream-downstream movements first filling reservoir

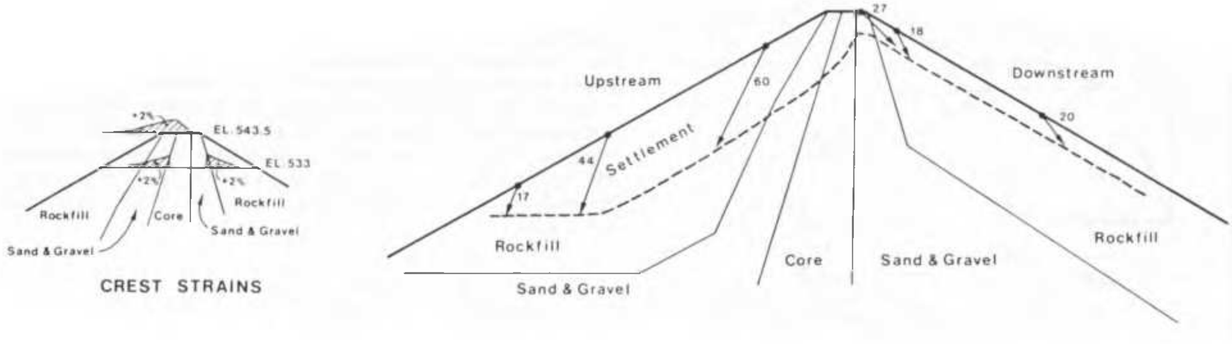


Fig 12. La Angostura Dam, Mexico. Surface movements during first filling

The possible consequences of such movements are:

1) Transverse cracking
2) Longitudinal cracking
3) Hydraulic fracturing
4) Arching and stress concentration
5) Development of plastified zones
6) Cracking of internal structures or conduits
7) Damage to instrumentation

Although the deformations that develop may not impair the safe performance of the embankment itself, the designer should be aware of them and take precautions to minimize the movements by:

1) Selection of materials
2) Proper zoning
3) Control of material placement by altering the water content and method of compaction
4) Control of rate of first filling
5) Observing the performance with instrumentation

Finally, Prof P R Vaughan discussed deformation behavior of dams built with rather wet clayey fills on clay foundations. A transcript of the tape recording of his presentation follows.

"We in Britain, of necessity, have developed the habit of building dams on clay foundations, often with rather wet clayey fills. High pore pressures are common and undrained failure during construction often controls design. This situation seems to be comparatively rare in the rest of the world and you may be more easily entertained by an account of problems that you do not have to share.

"Just to illustrate the problems, Fig 13 shows the stress analysis of a rather trivial homogeneous embankment with the filling foundation treated as a drained material. The stress ratio around a potential, deep seated slip surface is surprisingly uniform and low. As I suspect you all know, there is no risk of a deep seated, overall drained failure under these circumstances.

"Fig 14 shows the same embankment, but the filling foundation is here treated as a saturated, undrained clay. The shear stress is not uniform around a

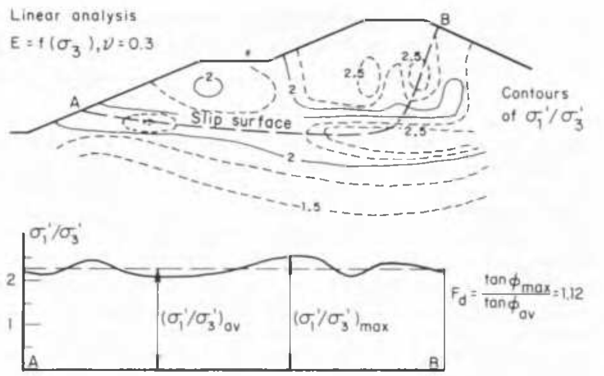


Fig 13. Homogeneous embankment with foundation treated as a drained material

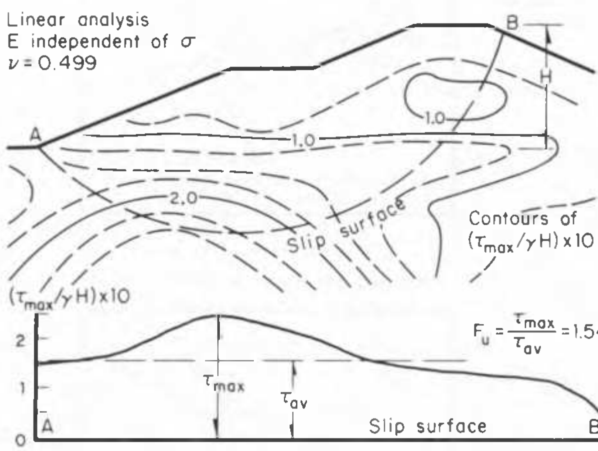


Fig 14. Homogeneous embankment with foundation treated as a saturated, undrained clay

typical slip surface, as shown at the bottom of the figure. There is a strong possibility of local failure or plastification occurring; there is a risk of progressive failure if the soil has strength-softening characteristics; and there is a risk of overall failure if the undrained strength is low or the embankment is high. I will discuss some aspects of this problem, and in particular, the important relationship between displacement and stability. Really, I am talking about a subject belonging to the second group of topics suggested by the Organiser at the outset.

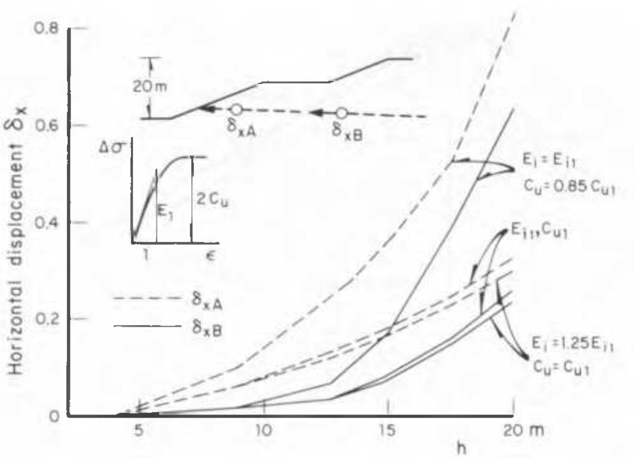


Fig 15. Non-linear displacement analysis of the embankment shown in Fig 14, treated as an undrained, saturated clay

"Fig 15 shows the non-linear displacement analysis of the same embankment, treated as an undrained, saturated clay problem (a final problem), and the displacements of two points on the foundation, A and B. The sensitivity of displacement to initial modulus and strength is indicated. You will see from the figures, that once local failure or plastification has ocurred, the overall deformation is controlled by the changes in strength to a much greater extent than by changes in initial stiffness. Under these circunstances, in fact, deformation is governed by strength rather than by the assumed deformation modulus of material.

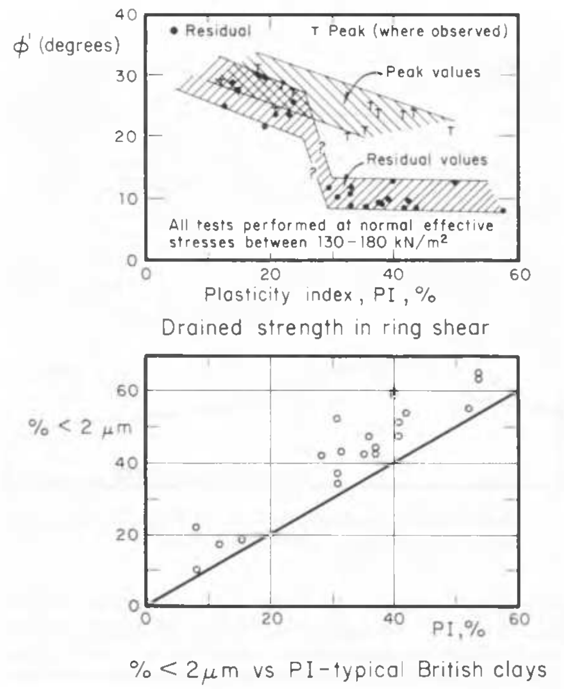


Fig 16. Drained peak and residual strengths of typical clays in Britain

"One of the controlling factors in this situation, is the nature of the undrained stress-strain behavior of the clay. Fig 16 shows the trend of drained peak and residual strengths measured in the ring shear apparatus, plotted against increasing clay content represented by plasticity. In the low plasticity range, the drop from peak to residual is typically small. The behavior is dominated by the coarse fraction of the grading and the overall behavior is that of a sand. But at higher plasticity the drop is large, platey clay particle behavior dominates, and slickensiding and particle orientation occurs.

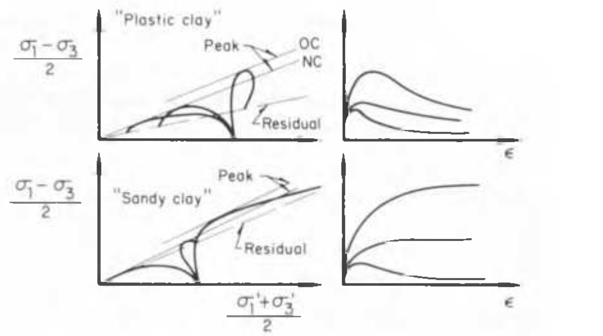
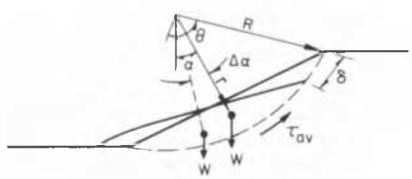


Fig 17. Undrained strength of intact clays. Classification by stress path

"Undrained behavior tends to match the drained behavior as shown in Fig 17. When the soil is equivalent to an overconsolidated soil, the sandy clay shows non-brittle, undrained shear, at least in the triaxial apparatus. The ultimate undrained strength is not reached until the stress ratio drops to something like the residual value. The more plastic clay, on the other hand, tends to show brittleness in undrained shear. However, the strains and displacements required to reach the residual value depend very much on the soil structure, tending to be much greater for the fill than for intact clay.



At failure

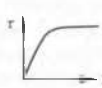
$W r \alpha \approx \tau_{av\,f}\, R.\theta$

After failure

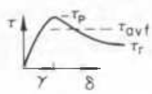
$\tau_{av} = \tau_r : W r\,(\alpha - \Delta\alpha) \approx \tau_r\, R\, \theta$

Hence $\Delta\alpha = \alpha \left(\frac{\tau_{av\,f} - \tau_r}{\tau_{av\,f}} \right)$

$\delta = R\,\Delta\alpha$



$\tau_{av\,f} = \tau_r = \tau_p : \delta = 0$



$\tau_p > \tau_{av\,f} > \tau_r$

$\delta = R\,\alpha \left(\frac{\tau_{av\,f} - \tau_r}{\tau_{av\,f}} \right)$

Fig 18. Post-failure movement of a slip surface (brittle soil)

"One consequence of brittleness is the post-failure movement of a slip, if one forms, as shown in Fig 18. The sliding mass moves to a new position where the shear stress required for stability matches the reduced strength resulting from displacement. In an unbrittle soil, on the other hand, the failed mass should be at a state of neutral equilibrium and there is no reason for a slip to move.

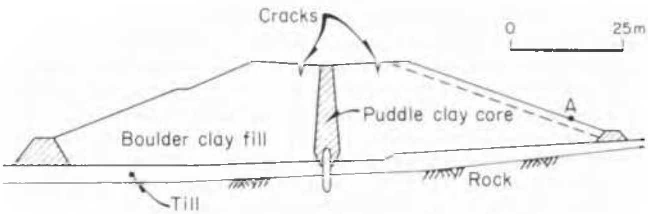


Fig 19. Cross-section of the Muirhead Dam when it reached the state of neutral equilibrium

"There is a reasonable amount of field evidence to show that this state of neutral equilibrium can develop in undrained shear of sandy clays. Fig 19 presents the Muirhead Dam built some 40 years ago, of a rather wet fill, and intended to be 27 m high. When the dam was 21 m high, large movements became evident and displacement measurements of the outer slopes were started. The old dam was built of glacial till.

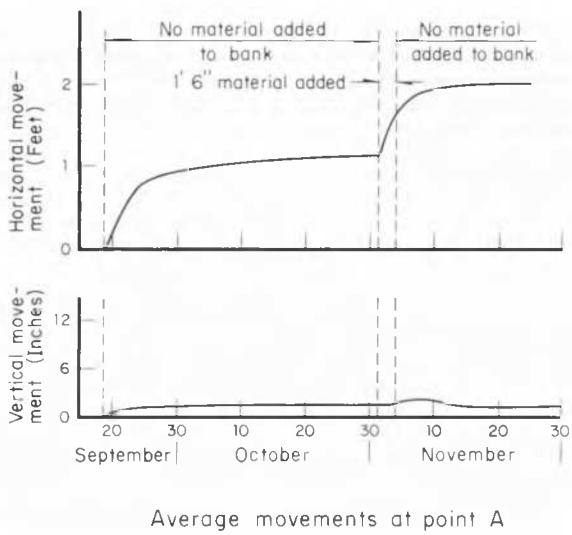


Fig 20. Horizontal movements vs time at Muirhead Dam

"The top of Fig 20 shows the horizontal movements of the dam plotted against time. When half a meter of fill was added, the dam spread roughly 0.3 m sideways and refused to rise any further. A true state of neutral equilibrium was reached. The engineers completed the dam at 21 m high as illustrated in Fig 21; it remains so to these days as a monument to the power of excess pore pressure.

"In this type of situation, the limiting condition is acceptable deformation, by definition, rather than slip. Deformation prediction is an essential element of design and deformation measurement is an essential feature of monitoring. It may also be noted, as shown in the previous figure, that in these materials the creep problem appears to be minimum.

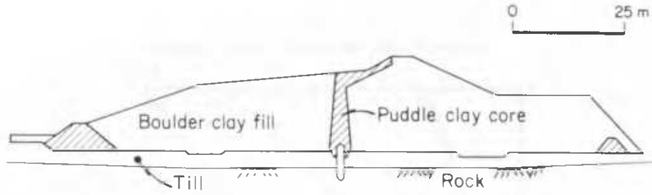


Fig 21. Typical cross-section of Muirhead Dam as built

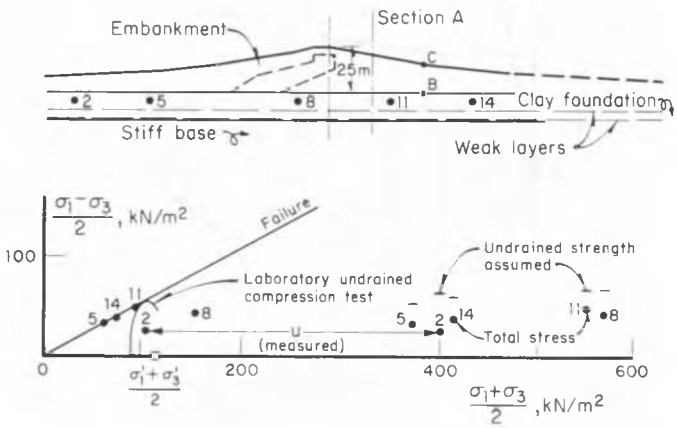


Fig 22. Cross-section of Empinham Dam and undrained shear strength of clay foundation

"If brittle-undrained shear occurs in a clay, then the behaviour is likely to be very different; local failure or plastification develops and a shear zone will form in a weak layer of the soil, where the overall strains and displacements are small. The influence of such a condition and the influence of progressive failure as well on the deformation of the dam is illustrated in Figs 22 and 23. The top of Fig 22 presents the section of the Empinham Dam with an undrained clay foundation. The influence of two weak layers have been studied by non-linear finite element analysis. The strength of the weak layers was assumed to be 2/3 of that of the surrounding clay. The predicted displacements of points B and C on the dam section are shown in Fig 23. Despite the large difference in safety factors when the weak layers are included and when they are excluded, the difference in displacement for the two cases is very small. In Fig 23 you can see the downstream movement of points B and C plotted against dam-height; up to two tenths, the movements are practically indistinguishable. The reason is that once the material has failed in the weak layers, the development of strains in the surrounding material is inhibited; large strains are concentrated in the weak layers, and strains are much smaller in the surrounding material; overall displacements are much the same in the two cases. The factor of safety in the weak layers case at the end of construction was about 1.2 and without the weak layers was of the order of 1.8. Really, it is very difficult to relate overall deformations and stability in such a case. Perhaps the only way is to use profiling techniques, such as inclinometer casings, to monitor the formation and propaga tion of shear zones. These casings can be located, from analysis, in the places where such zones are most likely to form. I will mention one last problem typical of plastic-brittle clays.

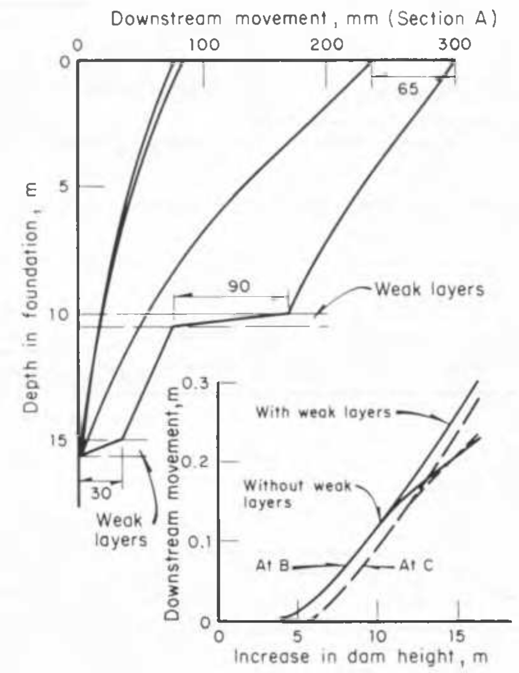


Fig 23. Non-linear finite element analysis at points B and C of Empinham Dam (see Fig 22)

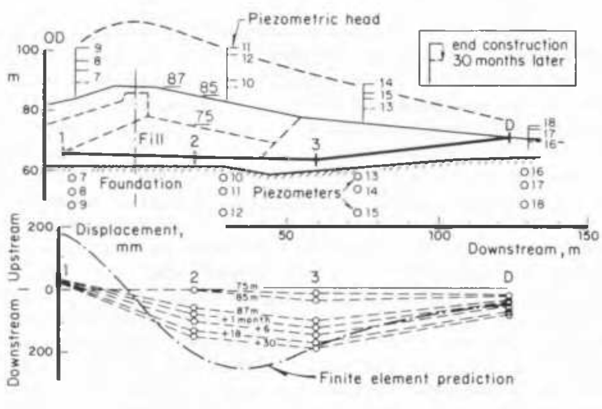


Fig 24. Horizontal displacements measured at Empinham Dam during and after construction

"Fig 24 shows horizontal displacements measured beneath the same section of Empingham Dam, during and after construction. A finite element analysis prediction based on soil data obtained from a trial embankment is presented in Fig 24. Prof Lambe might be interested to know that this is a class A prediction. At first sight, the agreement may look quite good.However, if you inspect more closely the figures, they will reveal that the deformation at the end of construction is only half those predicted by the analysis. The deformations more than double over the 30 months following the end of construction, by undrained creep, reaching values of 200 mm. A creep of this magnitude occurred when the overall factor of safety against yield of the structure, in terms of an effective stress analysis, was of the order of 2.0. This problem of undrained creep is a major problem in the plastic field, it probably dominates deformation behavior, and needs to be considered very closely when making deformation predictions".

After the presentation by the panelists, a brief discussion developed among them, related to *hydraulic fracturing*. Mr Wilson questioned Prof Alberro on the particular case of the Guadalupe Dam in Mexico, where both hydraulic fracturing (section 0 + 120) and cracking induced by differential settlement (section 0 + 260) occurred. Alberro noted that his case of hydraulic fracturing developed during drawdown without an increment of differential settlement and with generation of tensile effective stresses. These facts are based on measurements of total stresses, piezometric levels and horizontal strains.

Dr Vaughan remarked that hydraulic fracturing could only occur when the minor principal effective stress became tensile while reservoir seepage pressure was applied to the dam. The development of uniform seepage pressure within a dam would generally induce shear failure at points of low stress (Mohrs circles of stress being shifted towards the origin and becoming tangent to the failure envelope) before fracture could occur. Such a process would tend to increase average stresses and so might prevent fracture occurring.

Alberro concurred with Vaughan and indicated that the hazards associated with hydraulic fracturing can be avoided by means of protective measures, such as filters, adequate materials, proper specifications for placement, and so on.

Part II. DISCUSSIONS

The following contributions and questions from the floor were accepted for discussion.

Prof Z Eisenstein (Canada), "Pore pressures in a core during first filling".

Prof W Wolski (Poland), "Hydraulic fracturing and erosion".

Prof Y Ohne (Japan), "Evaluation of hydraulic fracturing".

Mr A D Hosking (Australia), question on "Correlation between predicted and actual transverse cracking in dams, and the tools necessary to predict such cracking". Mr Hosking also offered a contribution on "Some experiences from Talbingo Dam and Dartmouth Dam, in Australia" which could not be discussed in the Session.

Dr P Anagnosti (Yugoslavia), "The effect of transient seepage on hydraulic fracturing". Dr Anagnosti also offered two questions which could not be considered in the Session.

Mr G R Post (France), "Comparison of computed and measured crest deformations of several earth and rockfill dams".

Prof T Sawada (Japan), "Design considerations at the interface of a composite concrete and earth dam".

Dr V Radukic (Yugoslavia), "Deformation properties of soil as placed in a dam".

Dr M Maksimovic (Yugoslavia), "The use of influence coefficients for the computation of displacement and stresses". This contribution could not be presented verbally for lack of time.

Prof Eisenstein referred to experience at Mica Dam where piezometric measurements suggested that if was possible to analyse hydraulic fracturing by taking into account the effect of the external load applied by seepage to the upstream face of the dam, but without considering seepage within the core. In reply Prof Alberro said that, depending on the characteristics of the embankment, steady state seepage might take some time to establish itself. If this were so, he agreed with the proposition of Prof Eisenstein. Prof Alberro emphasised the need to take into account the interaction between shell and core due to the changing external forces, even for the case mentioned by Prof Eisenstein. Prof Eisenstein commented further that the case he referred represented one boundary condition and that considered by Prof Alberro in his presentation another boundary condition, the former being sometimes more critical than the latter. Mr Wilson pointed out that field observations show that the top of the core moves upstream during the first filling of the reservoir and that this fact contradicts the assumption made by Eisenstein (total load of water acts in the upstream face of the core). In his reply, Eisenstein mentioned that the upstream movements of the dam-crest are associated with the structural collapse of the upstream shell.

Dr Vaughan said he was in general agreement with the limiting cases presented by Profs Alberro and Eisenstein. However, he pointed out that cracking might often be due to local effects within the core such as changes in geometry, in material properties, and the inclusion of more permeable layers, which might not be taken into account in generalised analyses.

Prof Wolski commented that sharp changes in the geometry of an impervious core could lead to hydraulic fracturing, and that it was necessary to be very strict in both the design and construction of filters to avoid erosion and piping of the core, using model tests in design as necessary. He referred to his work (Wolski, 1970) on a simple method of checking filter criteria. Mr Wilson stressed his agreement with the importance of filter provision. He also asked Prof Wolski if the finite element analysis presented for the Hyttejuvet Dam was done *a priori* or *a posteriori*. Prof Wolski confirmed that the computations were made *a posteriori*.

Dr Vaughan said that since the importance of filters as protection against the consequences of hydraulic fracture, or any other form of cracking, was clearly agreed, the topic of filter design and provision would be suitable for a speciality session at the next international conference.

Prof Ohne indicated that two equations had been used to evaluate the risk of hydraulic fracturing, namely:

$$\sigma_3 + |\sigma_t| < p_w \qquad (1)$$

$$\bar{\sigma}_{xf} = \bar{\sigma}_{xi} + i\gamma_w \quad ; \quad \bar{\sigma}_{zf} = \bar{\sigma}_{zi} - \gamma_w h \qquad (2)$$

where

σ_3	minor principal stress
σ_t	tensile strength
p_w	hydrostatic pressure
$\bar{\sigma}_{xi}, \bar{\sigma}_{zi}$	initial effective stress in x and z directions
$\bar{\sigma}_{xf}, \bar{\sigma}_{zf}$	final effective stress in x and z directions
i	hydraulic gradient
h	hydraulic head
γ_w	unit weight of water

Prof Ohne pointed out that, from equation 1, a shear failure would occur, rather than hydraulic fracture, if

$$\sigma_1 > q_u$$

where

σ_1 major principal total stress, q_u = unconfined compression strength

According to equation 2, in effective stress, shear failure would occur when the Mohr's stress circle touched the failure envelope. Prof Ohne considered equation 1 to be acceptable provided $\sigma_1 < q_u$. Prof Alberro expressed general agreement, and pointed out that both expressions defined hydraulic fracture as occurring when tensile effective stresses developed. Dr Vaughan commented that shear failure preceding hydraulic fracture could have a major influence on the stress distribution.

Mr Hosking referred to measurements at two Australian dams (Talbingo and Dartmouth), and inquired about the existence of new correlations between the prediction and observation of transverse cracking in embankment dams, and about current views on the type of test required to make predictions. Mr Wilson replied that attempting to make such predictions was extremely uncertain. Deformation rate played an important part in determining whether cracking occurred. He cited El Infiernillo Dam, where transverse cracking developed during first filling when the maximum crest extension was 0.3 per cent. In subsequent years, after the cracks had been filled in, the strains slowly reached 1 per cent without further cracking.

Prof Anagnosti referred to the ideas expressed by Eisenstein and Alberro, and sugested: First, that the general stability of the dam was not affected by assuming either full reservoir pressure acting on the upstream face of the core or steady seepage through the core. Second, he said that intermediate transient stages of seepage should also be considered in relation to hydraulic fracture. Prof Alberro and Dr Vaughan agreed with the second point but pointed out the difficulties of taking transient flow into consideration.

Mr Post referred to the measurements taken at the crests of the Youssef Ben Tachfine and Moulay Youssef Dams, which enabled previous computations that had indicated potential cracking zones to be checked. Mr Wilson and Dr Vaughan accepted that predictions allow the determination, in a very general manner, of zones where cracking was most likely, and where protection in the form of suitable filters to prevent crack erosion was most desirable.

Prof Sawada discussed the design of joints between concrete and embankment of a composite dam.

Mr Radukic referred to deformations observed at Poechos Dam in Peru, where a comparison was made between computed and measured displacements. The difference between them probably resulted partly from creep and partly from non-linear behaviour.

The session was then declared closed, and the Organizer expressed his thanks to the participants for their attention and to the panelists and to the constributors from the floor.