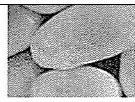

Range of Materials Vulnerable to Liquefaction Flow

Having checked the hypothesis introduced earlier against laboratory data, and having found it instructive in considering earthquake liquefaction, it will now be applied to the question of whether or not a category can be found for deposits which are vulnerable to liquefaction, and thereby provide a rational basis for developing a tool which could be optimised to treat only those types of materials.

Before launching an attempt to discriminate between natural and artificial deposits which could, and which could not liquefy, it is necessary to acknowledge that the hypothesis presupposes two characteristics of the mass: that it be fully water saturated, and that it be non-cohesive, that is, composed of discrete particles. In order to preserve the generality of what follows these limitations need to be reviewed now. In the context of liquefaction, the terms saturation and discreteness are interrelated through apparent cohesion, that is, the component of strength derived from surface tension in the menisci between particles in partially saturated soils. One reason full saturation is synonymous with the definition of liquefaction in common use is that apparent cohesion cannot exist under this condition. Deposits which possess the more permanent type of cohesion, derived from the surface activity and adsorbed water associated with clayey minerals, are generally considered to be safe against liquefaction. Here that confidence is attributed to cohesion's ability both to hold even a loose structure together following a jolt, and to prevent individual particles from falling freely in the manner necessary to generate ***epwp.*** Consequently, it is believed the limitations of saturation and discreteness contained in the hypothesis place no inherent constraints on the following assessments.

PARTICLE SIZE

The values listed in Table 3 suggest that the smaller the grain size, the more prone to liquefaction a mass of discrete particles would be. So in order to place a lower limit on liquefiable grain size, if one is appropriate, it becomes necessary to associate discreteness with size. The lower bound of particles which can exist as discrete grains is probably natural silts of quartz origin, or artificially produced rock-flour (tailings). This is because material containing even a small proportion of clay sizes is generally found to be cohesive. This in turn is due to the fact that clay size particles have the high Surface to Volume ratios necessary to make surface activity significant. In addition, in natural deposits these sizes are generally derived from the feldspars and similar "clay forming" minerals prone towards water adsorption.

The upper bound of particle sizes considered liquefiable is here set at fine gravel. This limit comes from consideration of the implications of the relationship plotted in Figure 14. For two reasons, gravel size particles are denied the opportunity to generate high ***epwp***: the distance gravel needs to fall in order to approach V_T is not available within the void space of even a very loose mass; and, even if adequate fall distance were available, the inherently high ***k*** of gravels (uniform) can allow the void water to vent at the maximum possible rate of structural collapse (V_T) without necessitating any more than an insignificant gradient. Consequently, gravel sizes can neither be carried in suspension, nor can they become part of a flow.

PACKING DENSITY

Density of packing is a determining factor in liquefaction for several fairly obvious reasons:

- The looser the structure the more easily can it be made to collapse.
- The looser the structure the better the opportunity for dislodged particles to generate higher ***epwp*** since the speed of their fall has more room to build up.
- The voids of loose assemblages contract during forced deformation, whereas voids in dense masses try to dilate.

In Table 4 it may be seen from comparing the values in Row A with Row C that as the packing changes from loosest to densest, the void space decreases from 0.91 to 0.35, while the **s:w** ratio increases from 1.1 to 2.9, and the number of contacts per particle increases from 6 to 12. Although these values are for idealized arrays, a generalization of this inference is credible: if the volume of solids exceeds the volume of water by more than a small amount, say 1.5:1, there will be a kinematic problem finding the space to allow particle movements. To perhaps a lesser extent, the stabilizing effect of having the redundancy of inter-particle contacts existing in a denser mass is advantageous. In all, it seems reasonable to conclude that the looser the density the more prone to liquefaction the mass is.

The geological opportunities for a loose cardhouse packing to develop and survive in a cohesionless deposit are possibly linked to special circumstances of deposition. While it is believed that sedimentation from laminar flow will form a relatively dense structure, it is entirely possible that particles coming out of suspension where upward vectors affect the stream bed would be looser and more poorly organized. Such flow vectors can arise where channel geometry and other hydraulic factors promote non-laminar or turbulent flow. Perhaps the best natural opportunity to aggregate into a cardhouse structure is where aeolian deposits form in moist environments.

Denser masses, apart from being associated with deposition from streams, as discussed earlier, are also likely to form on slopes. Inclined surfaces are a *de facto* shear environment, and as a consequence, sedimenting particles must adopt a sufficiently stable structural configuration during deposition to accommodate some degree of shearing. Deposits precondition to shear in this way are denied the opportunity to exist in the loosest state. Gravel sizes, and larger, which are normally deposited on steeply inclined stream beds would seem to have a natural predisposition towards a dense structure. Additional factors which favour denser arrays in gravels are shown in Table 3 where the cardhouse promoting suction forces are insignificant, and the sedimentation energy favouring close packing is large.

Earthfills which are placed and compacted in the dry are normally designed with the specific intention of avoiding severe deformation under loading, and are normally intrinsically stable. But where tall fills are compacted dry of optimum, and are subsequently saturated, they may stope, leaving the upper levels in a loosened state. Dredged fills, when they are discharged underwater as a turbulent flow, or subaerially as a heavy slurry, can be vulnerable because these modes do not promote dense packing. Tailings, depending on how they are placed, can have essentially the same potential problems as dredgates.

MASS GRADATION

Geotechnical intuition is strongly inclined towards believing that the load-deformation behaviour of well graded materials is intrinsically superior to poorly (uniformly) graded materials. The information listed in Table 4, where $U_C = 1$ for both the loosest (A) and the densest (C) packings seems to contradict this position, suggesting that gradation is a bad indicator, and that the volume of void space is far better at predicting the deformation response to loading. The fact that low void ratios are synonymous with good gradation could perhaps be the subliminal basis for the intuition. So performance may come down to amount of space available for particle movement during loading—the less space the better. The question would then be: what is space required for?

According to the hypothesised mechanism, response to loading involves a tendency towards particle rotation, which in turn involves some amount of asperity truncation. Table 4 shows a general trend towards a greater number of inter-particle contacts (**#i-pc**) with increasing U_C, thus indicating more kinematic resistance with improved gradation. The hypothesis also suggests that ***epwp*** generation is dependent on the degree to which dislodged particles achieve V_T during their fall. Of the gradations depicted in simplistic terms in Table 4 only A and B provide any real opportunity for particles to fall at all, whereas C, D and F call for particles to rise in response to deformation.

Row E represents a loose matrix within a loose skeletal structure formed of larger particles, and its response is described as contractive. But the behaviour of E differs from that of the other contractive structures A and B in that it will not lead to the full $\boldsymbol{i}_{MAX}$ condition during collapse. This is because the skeletal particles are prevented from falling freely by the matrix, and although the matrix itself, dependent upon its relative size, may achieve full V_T, the overall ***epwp*** experienced by the mass will be limited to that proportion of mass volume occupied by the matrix. In the particular mix represented by E, where the porosity of the skeletal structure is 0.48, this would limit the internal hydraulic gradient during collapse to 48% of $\boldsymbol{i}_{MAX}$ for the whole mass.

It is possible to pursue the behaviour of skeletal-matrix masses such as E into the realm of well graded materials such as sandy gravel, but this is hardly necessary. The point is this: when a soil is composed of more than one size particle, the smaller particles might have the room to fall far enough to reach V_T, but these smaller particles will get in the way of the larger particles and prevent them from reaching their V_T. In the case of a well graded soil, where particle sizes vary greatly, the fraction of the soil which can achieve V_T will be correspondingly small, and any internal gradient set up during failure would be proportionately small.

The conclusion therefore is that it is only in a very uniformly graded soil that all particles can achieve V_T simultaneously, and as gradation improves, an increasingly smaller percentage of the saturated mass has the room to fall into suspension.

GRAIN SHAPE

To what degree, if any, particle shape makes a deposit more or less prone towards liquefaction is a question which can only be approached here by comparing the two shapes discussed so far, those being, spherical and ellipsoidal. The following general comparisons are made by equating an ellipse whose diameter normal to the elliptical shape is equal to that of the sphere. This accords with the geotechnical standard for measuring size, since both sieve and hydrometer analyses are insensitive to the longest dimension. Nevertheless, it is worth remembering that by this definition an ellipsoid with 2:1 aspect ratio is twice as heavy as a sphere of the same size.

Spheres, since they are inherently unstable laterally are less likely to form a loose structure than are ellipsoids. In addition, because ellipsoids have smaller radii of curvature at their ends, they are more exposed to high surface tension forces than spheres, and consequently, more prone towards cardhouse structure formation. On the other hand, because ellipsoids are heavier than spheres, the ***epwp*** generated for any fall of an ellipsoid would be less than for a sphere of equal size. Finally, in the case of a particle exposed to lateral flow, an ellipsoid is more stable than a sphere because friction depends on weight, and drag on diameter. So, it seems that both shapes have stability advantages which depend on the geotechnical context, and any consequence of shape would have to be assessed on a site specific basis.

CONCLUSION

Applying the hypothesized mechanism to the problem of identifying which deposits are prone to liquefaction leads to the conclusion that only those deposits which adhere to the following conditions are vulnerable:

> They must be saturated, uniformly graded, non-cohesive, of silt or sand size, and have been deposited in an environment which promoted a very loose (cardhouse) structure.

It must be confessed there is nothing new in this conclusion, and if truth be told, Arthur Casagrande said essentially the same 25 years ago.

Nevertheless, the hypothesis gives some fundamental suggestions as to a framework within which the specific geology of an individual site may be interpreted in a way which can help decide whether a deposit needs to be densified or not.

Interpretation Solution

The simple success in treating clean sand inside the Molikpaq was followed later by an ambiguous outcome in treating the fine tailings at Myra Falls. Earlier efforts to understand the results at Myra Falls were inconclusive mainly because of the inability to explain why the volume of water discharged from the exhaust was so small in comparison with the volume of the ground depression. This concern was based on intermittent visual estimates while the author was observing the operation. Difficulty in interpreting the Myra Falls field data in a convincing manner created the need to develop a working hypothesis to explain the results, and perhaps, to decide on the actual value of that particular effort. The hypothesis, as developed here, is believed to be internally consistent and compatible with physics and geotechnical principles. It also seems to be sufficiently robust to explain laboratory liquefaction results, and to offer insights into earthquake liquefaction; consequently, it will now be used to revisit the Myra Falls data.

On the general principle that if something is to be learned about soil behaviour it is most likely to be found by looking into what is going on in the water phase, the pore pressure measurements recorded by the CPT probing were re-examined in detail. The CPT used at Myra Falls was of the piezoelectric type and recorded Dynamic Pore Pressure Response ("DPPR") at 50 mm intervals. This data formed the basis for the plots described earlier (Figure 11). In addition, at rod changes, while the probe was stationary, the pore water pressure was automatically logged at 5 second intervals. The history of four such records from CPT-30 is shown in Figure 16. Here it may be seen that in three of the four histories the DPPR was very close to full vacuum

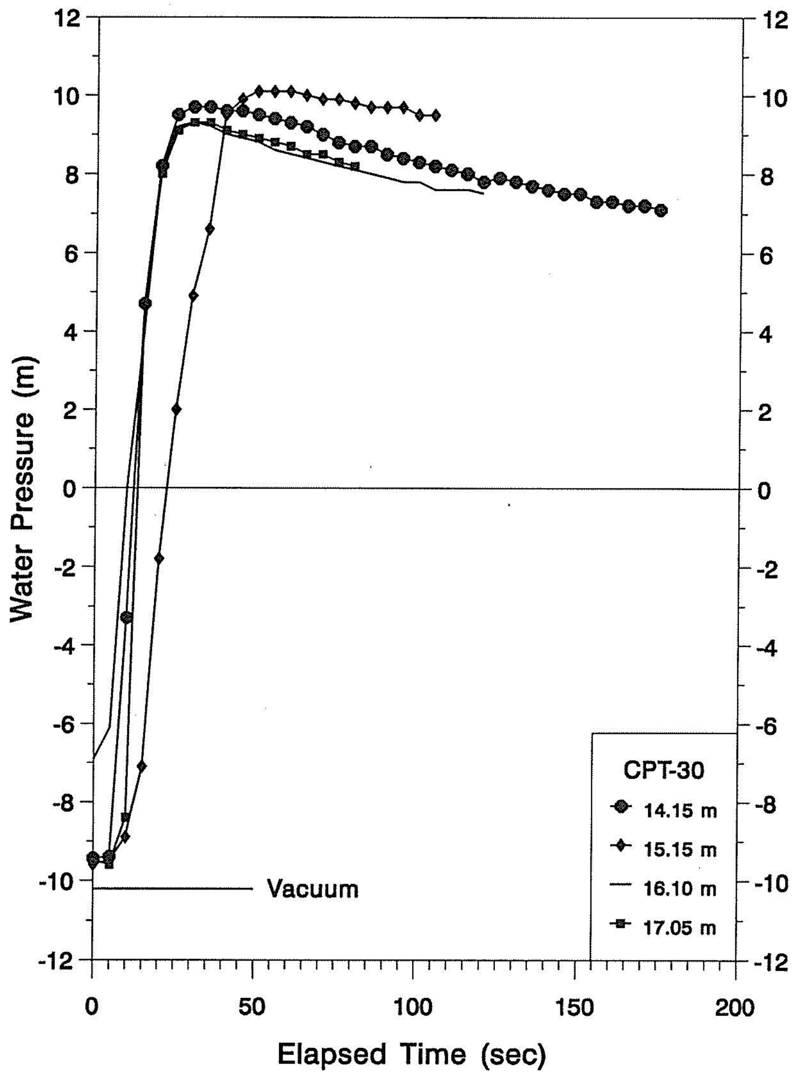

Figure16. Myra Falls DPPR Dissipation Histories

when penetration was stopped. The pore water pressure then rebounded to a peak value and subsequently drifted towards the ambient pore pressure at that level within the tailings. In eight of the 20 such histories logged in CPT-30 sufficient time was allowed for the records to stabilize, giving the equivalent of piezometric head at the CPT tip elevation. These 8 values are plotted on Figure 17 as indicated. An explanation of the rest of the information on this graphical summary follows:

Line A	is the best fit curve for the 8 stabilized DPPR dissipation points, and indicates the upper bound piezometric head profile through the tailings.
Line B	represents the water pressure profile which would exist for hydrostatic conditions within the tailings, that is, if the water was in fact static (not flowing).
Line C	represents the fluid pressure profile which would exist for the condition where the deposit had collapsed into a suspension, as in liquefaction.
Line D	is the upper bound of DPPR values recorded before treatment in CPT-12, and which were shown as shaded background in Figure 11.
Line E	is the trace of DPPR recorded after treatment in CPT-30.
Line F	is the absolute boundary of negative pore water pressure that is, full vacuum.

Line B is the condition which exists in a deposit where there is no ***epwp*** and no hydraulic gradient. The fact that Line A is not coincident with Line B can only be explained by flow occurring within the tailings impoundment. The slope of Line A shows an increase in pressure of only 0.51 metre H_2O per metre of depth increase, whereas the static condition is unity. Therefore, an externally applied component of vertical downward gradient equal to 0.49 must exist through the tailings.

Line C is the maximum ***epwp*** which could exist if the full depth of the tailings collapsed into suspension instantaneously. The fact that

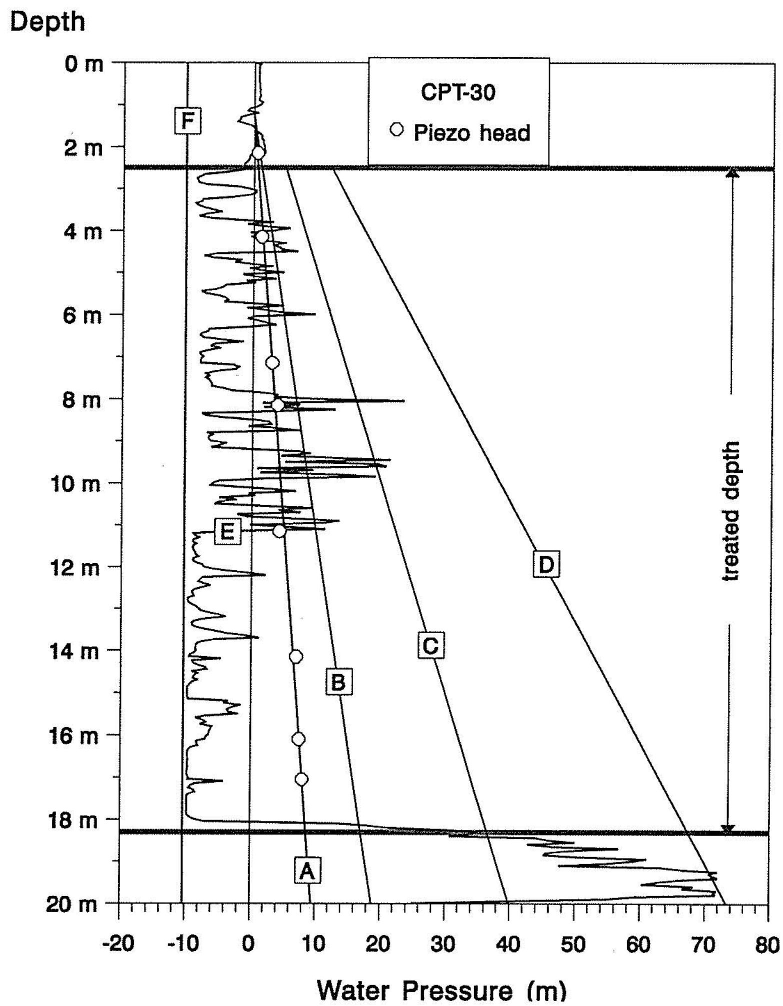

Figure 17. Myra Falls DPPR Interpretation

the DPPR recorded before treatment exceeds this "limit" by more than 80% is because Line C is for the idealistic one-dimensional case of horizontal ground, and Line D is for the axi-symmetric failure around the CPT tip as it pushes through the ground. The local soil pressures built up as the cone causes rapid punching (deep pile bearing) failure can obviously exceed total overburden pressure. The CPT penetrates the ground at 20 mm/s, and in terms of velocity, this is equivalent to V_T for a sphere of D=0.2 mm; consequently, any saturated loose uniform mass composed of particles smaller than fine sand, as was almost all of the tailings, would be driven into suspension around the tip.

Line E only exceeds the hydrostatic pressure (Line B) rarely, and even then, only at spikes in the trace. A large majority of the DPPR is in fact negative to a significant degree. What is most striking is the proximity with which Line E approaches the absolute minimum limit (Line F), especially below 11 m depth. According to the position developed earlier, ***npwp*** is triggered immediately a saturated mass tries to increase its volume at a rate faster than seepage inflow can occupy the larger space being created. What is evident from Line E is that the demand for inflow could not be supplied at the rate the tailings was dilating, and this is despite the fact that the volume being forced to expand was confined to a few cubic centimetres surrounding the CPT tip, and the recharge area was spherical.

At a depth of 11 m a distinct change is obvious in the DPPR response of the tailings depicted by Line E. To account for this change the site data was reviewed. CPT-12 and SPT-4 data from the pretreatment period do not show a change in penetration resistance at this depth, although the material below about 15 m appears somewhat denser. Grain size analyses from borehole samples are shown in Figure 6, and no discernable trend can be seen there either. Some variation was then sought using the UBC Simplified Soil Behaviour Type correlations as advocated by Robertson & Campanella (1984); this manner of approximate discrimination between soil types is shown on Figure 18. Again, no significant change in material type could be detected across the 11 m depth, although the lower tailings tended to be stronger. So, in the absence of obvious density or gradation

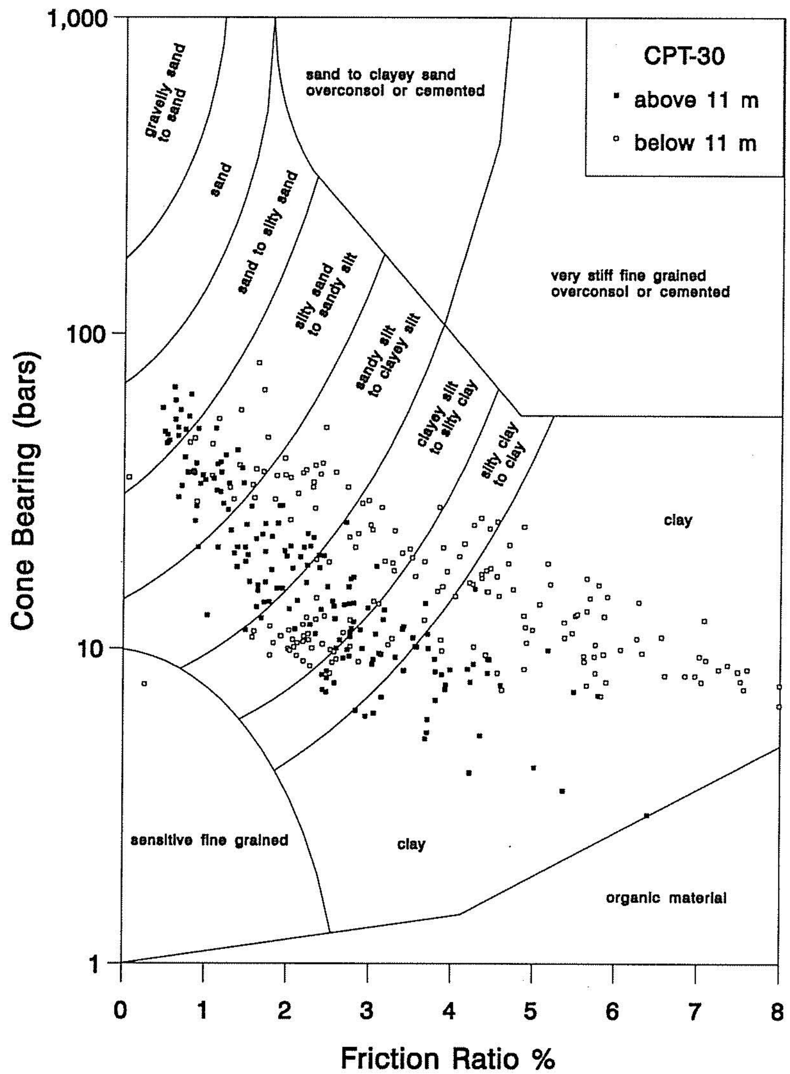

Figure 18. Myra Falls UBC Soil Type Interpretation Chart

changes, it is speculated that the abrupt behaviour change at 11 m is a result of varying performance of the Vibro-Drain above and below that depth. This might be explained by the following circumstances:

At the beginning of each treatment, the vibro-draining equipment was inserted through the full depth of tailings, with this depth being evidenced by refusal of the stinger as it encountered the underlying drainage blanket. Treatment then progressed upwards in a step-wise progression. The initial vertical intrusion through the layered tailings would have created a disturbed column in which the vertical permeability would have been significantly increased compared to the generally layered tailings elsewhere. This column of preferential seepage, in combination with the 0.49 downward site gradient (noted earlier as being superimposed on the static environment) could have facilitated flow of excess void water down to the under-drain. Water, discharging into the tailings under-drainage system, rather than exhausting through the filter/drain module, would help explain the scarcity of water seen at the surface. The fact that water might find it easier to descend to the under-drain, instead of entering the filter/drain module, would not be plausible were it not for the fact that there was evidence of icing in the exhaust conduits. This became obvious when gravel size ice particles were occasionally ejected with the discharge, and frost was seen on the outside of the filter element. To account for the better performance at depth, by claiming that downward flow made up for the shortcomings of the filter/drain module, makes it necessary to further speculate that at about the 11m depth significant downward flow was cut off either because of remoteness from the under-drain or due to caving of the weaker tailings above that level.

The final, and most important matter of interpretation still remains, and that is: was the degree of improvement achieved at Myra Falls, and as defined by the site investigation data reported here, sufficient to prevent liquefaction of those tailings? The tailings consisted of a mixture of silt and sand sizes with a considerable quantity of clay sizes as indicated in Figures 6 and 18. The presence of clay sizes in these tailings is not believed to impart cohesion to the mass, as it would suggest in a natural deposit, but is rather a consequence of the milling process which creates rock flour. Such sizes are here considered surface-inactive, at least in the short term. Therefore, what is involved here is a matter of dealing with a loose saturated mass composed of discrete particles which exist in a multitude of layers, many of which could be uniformly graded. Consequently, the pre-treated Myra Falls tailings fulfil all the prerequisites for being vulnerable to liquefaction failure.

A comparison of the before and after penetration tests (Table 2) show a significant improvement in both SPT and CPT values. Also, a large surface depression developed in the area of treatment while the work was in progress, the true magnitude of which was masked by the presence of a geotextile mat within the test pad fill. This evidence alone establishes that the original loose structure was made to collapse and adopt a closer packing. According to the arguments set forth above, causing the loose structure to collapse in a manner where the vibrations persist until the surplus water has been vented is treatment enough to take the deposit out of danger of subsequent liquefaction. Of course, this may not be enough work to achieve the particle packing and/or orientation needed to make it respond in a strain-hardening way, but this is of little importance in a normal tailings pond.

The most compelling evidence of the adequacy of the treatment is undoubtedly the DPPR data (Line E). Here it is undeniably evident that when the treated mass is forced to deform it experiences an extreme water demand, a demand closely approaching the physical limit of water. It is therefore considered to be manifestly impossible for a mass in this state to collapse into a water suspension, and flow as a fluid when forced to deform under transient loading.

The only style of argument that the author can anticipate being levelled at this conclusion is one which would invoke the cyclic mobility type of assumption. Such a contention might run along the following lines:

> "Negative DPPR is evidence of an attempt by the structure to dilate, with the greater separation of the particles being accomodated by larger void spaces. The inward gradient accompanying dila-

tion creates flow from outside to fill this increased void space with water. Eventually, this process results in sufficient water enhancement to accommodate a liquefiable structure."

It is only the third and final sentence which is wrong, and it can be ruled out for two reasons:

(a) The inward gradient will persist only long enough to allow the required dilation, after which time it will become zero. In other words it will not follow, nor drive, the structure into the contractive range.

(b) The depth to which water can be supplied for void enlargement is dictated by the duration of the deformation cycle and the ***k*** of the mass. For an earthquake Shear-wave the time limit is about 5 seconds. At Myra Falls this would limit ingress to about 20 mm from the surface, and then, only in those areas of the impoundment where standing water was ponded above the tailings. Any water entering the surficial mass in this way would have an equal opportunity to leave the mass during the reverse cycle.

It is concluded, therefore, that there is simply no conceivable way in which a large body of this sort could be induced to fail by liquefaction, especially within the time frame of an earthquake, after being treated in the way, and to the extent, it was.