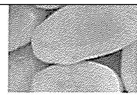

Published Laboratory Data as Check on Hypothesis

The purpose of the following is to determine if the mechanisms suggested in the last section are compatible with the observed load-deformation behaviour of saturated sand specimens published by others, and to see if perhaps some of the as yet unexplained features of those results could be interpreted in terms of this hypothesis. Although reference is made mainly to triaxial compression testing, essentially the same results can be elicited by monotonic simple shear or cyclic loading.

Liquefaction has been observed in laboratory triaxial shear testing of saturated sand. This mode of response is one where, after a small amount of deformation, the strength collapses to a small fraction of the peak strength, and there is no subsequent recovery of the strength with further straining. This degree of liquefaction has only been achieved in compressive testing where the specimens have been prepared by "moist tamping". As discussed earlier, making specimens from moist sand promotes the formation of a cardhouse structure where one particle can be propped up in a semi-vertical position by a similarly non-horizontal particle of complementary orientation. Subsequent saturation of the specimen then obliterates the surface tension forces supporting this sand skeleton, leaving a structure which has lost a vital part of its equilibrium. During compressive straining a crisis arises when some particles suffer sufficient promontory damage that they can rotate into a more aligned orientation normal to the major compressive stress direction. The void water which is expelled then rises to the top of the specimen where it starts to jack the loading cap off the sand structure, with the result that the water carries more of the deviator load itself. The consequent hydrostatic increase in water pressure is felt on the inside of the cylindrical membrane where it acts to partially negate the lateral structural confinement. This increases the effective principle stress ratio. Catastrophic failure of the mass is then inevitable as there is no rectifying influence to prevent the zone of structural collapse enlarging without restraint, and in doing so, releasing more and more free water within the specimen enclosure.

Temporary Strain Softening, a second mode of behaviour, involves significant loss of strength following an early peak, but in this case, the strength recovers with further straining, eventually exceeding the initial peak. This can be observed in specimens where the sand is "pluviated" through water to simulate natural fluvial deposition. Although this procedure promotes specimens in which the grains will accumulate with their long axes predominantly horizontal, the orientation of these flat lying grains will be random, since there is no stream flow to give them a preferential alignment. Also, the grains will come to rest without the horizontal component of kinetic energy associated with sedimentation from a stream. The laboratory specimen will therefore simulate natural fluvial deposits inasmuch as it will have flat lying grains, but it will differ in that the grains are less organized and less tightly packed in the horizontal plane. During compression such specimens would be expected to go through an initial crisis where particles, perhaps only those which had been reoriented during the consolidation phase, rotated again in response to the straining. In this case only limited water would be liberated as the structure experienced a minor change from point-contacts to body-contacts, and be manifest in only limited strength loss. Further straining then would result in mass shearing where grains rode over underlying grains and the dilation associated with this behaviour would draw water back from the loading cap and manifest itself in strength rejuvenation.

Strain Hardening, a third form of response, involves continuous increase of strength with increasing deformation. This behaviour is observed when specimens are prepared by pluviation and then compacted by external vibration to reduce void ratio. Here particles are already close to, or already in, the full body contact configuration.

Even in this case a transitory crisis point may be seen, and again, this may be associated with grain reorientation or asperity crushing in the horizontal plane during specimen consolidation. This temporary softening is most apparent in specimens which are consolidated under high confinement. In this regard it should be noted that a confining pressure of as little as one atmosphere can produce as much as 1% lateral strain in a loose array of angular particles simply by point crushing, and without leaving any evidence which could be detected by mechanical analysis. After this brief hesitation the strength continues to increases as dilation pulls water back from the loading cap.

Here, rather than considering specimen strength, which is dictated by the degree to which the membrane remains in contact with the sand structure, attention is focused instead on frictional resistance which is independent of the magnitude of pore pressure, or of how it is generated. The fundamental measure of the soil structure's ability to carry load at various phases of its deformation is the ratio of principle effective stresses; for convenience the equivalent concept of "friction angle" ø will be used. Researchers have identified three separate ø angles which appear to be constants for a particular sand (in compression) and these are listed below, together with the way in which these constants are interpreted in terms of the hypothesis:

$ø_{CSR}$ is the value measured at the initial crisis (peak deviator load), immediately preceding liquefaction or strain softening. This is believed to be the point at which asperity breakage allows particle rotation to take place.

$ø_{SS/PT}$ is the value measured during liquefaction (Steady State condition), or during temporary strain softening at the point of Phase Transformation. This is believed to be a measure of the resistance to particle movement while the grains are mutually aligned and oriented in the direction of local mass movement.

$ø_{MAX}$ is the maximum phi obliquity for the sand. This is believed to be a measure of the resistance to mass interaction (grain overriding) while movement is occurring parallel to the failure plane.

For examples of the relative magnitudes of these parameters, reference can be made to Vaid & Chern (1985) where values are given for both natural sand and tailings. Typically, $ø_{CSR}$ is somewhat more than half of $ø_{SS/PT}$, and quite remarkably, $ø_{SS/PT}$ is only marginally less than $ø_{MAX}$. Remembering that ø is the measure of the sand's capacity to carry load, this implies quite unambiguously that during laboratory liquefaction the structure has a frictional capacity ($ø_{SS/PT}$) close to its maximum undrained value ($ø_{MAX}$). Liquefaction "flow" in the laboratory must therefore involve a soil structure which is essentially intact. The depiction in Figure 15 of the re-deposition of a suspension following collapse is believed to be analogous to what happens during liquefaction in the laboratory specimen. After the loose mass collapses under $ø_{CSR}$ the frictional resistance improves to $ø_{SS/PT}$ because the mass reconstitutes itself in a denser packing. If the excess water were allowed to drain at that time it is likely that resistance close to $ø_{MAX}$ would then be available.

Vaid and his co-workers, eg, Vaid & Thomas (1995) have emphasised the observation that although $ø_{SS/PT}$ and $ø_{MAX}$ are essentially constants irrespective of testing procedure, $ø_{CSR}$ varies with confining pressure when tested in triaxial extension. Also, and this is later referred to with regard to earthquake triggered liquefaction, it is possible to elicit liquefaction behaviour in extension where the same specimen would show little if any contractive behaviour in compression. These laboratory results are interpreted here to indicate that extension tests on pluviated specimens produce a condition where the major principal pressure is in a direction parallel with the particles' longer axes, making the response both softer and weaker.

A fourth type of response, referred to as "cyclic mobility", is an arbitrary definition of specimen behaviour in the laboratory. As such it cannot provide much in the way of fundamental understanding of inter-particle behaviour. The testing involves numerous strain reversals, while the plane on which the major compressive pressure acts is continually changing. Eventually, the specimen passes through a transient state in which both of the effective principal stresses are simultaneously zero (which indicates the membrane has completely lost contact with the soil structure), and "liquefaction" is subsequently deemed to have occurred at some prescribed strain. In light of the hypothesised mechanism it seems that under these testing conditions the "effective" void ratio within the specimen could be radically altered by particle reshaping, as well as by fluid volume increase brought about by differential thermal expansion and by gas coming out of solution. Strength data generated by this form of testing may have application in the field at the interface between buried structures and the ground, but otherwise are considered here to be an artifact of testing. It is perhaps worth remembering that this term was coined by Casagrande (1976) in reference to his concern that cyclic testing suffered from the problem of water migration to the top of the specimen.

It appears, therefore, that the proposed hypothesis is compatible with published laboratory work, and furthermore, it offers some possible insight into the counter-intuitive laboratory findings which show an apparent increase in specimen susceptibility to liquefaction with increasing confinement.

Earthquake Liquefaction

Since earthquakes and vibro-draining both affect the behaviour of saturated sands by vibrations, it seems appropriate to examine the manner by which earthquakes bring about liquefaction in the hope of learning from, and perhaps imitating, that very effective natural process. First, earthquake mechanics will be looked at from the perspective of the three main vibro-draining design issues, which are: How to penetrate to the required depth; how to go about rearranging the particles; and, how best to manage the water? Then, two related phenomena which can follow liquefaction of a deposit will be addressed, and these are: bodily translation of the liquefied mass; and, water extrusion from the ground, and/or, the formation of sand boils.

TRIGGERING MECHANISMS

The first question, of how to get the energy source to the required depth, seems trivial in the case of earthquakes. But maybe it is not. The prevailing opinion is that liquefaction is brought about by the Shear-wave as it rises through the ground from the top of the buried rock. If this is so, the effect of the Shear-wave, which cannot be transmitted through a fluid, would be cut off at the first layer it succeeded in liquefying. Thereafter, liquefaction of the overlying ground could only progress by the mechanism of stoping. Stoping will be discussed later.

The second question, of the energy form best suited to stabilize the structure, may offer some collateral insight into penetration depth too: Triaxial work shows liquefaction can be triggered by purely static variation of vertical pressure, and Vaid and his co-workers have demonstrated that decreasing the vertical pressure (elongation) is more potent in bringing about collapse of the structure than increasing the vertical pressure (compression). This suggests that seismic waves which produce vertical heaving of the ground would be likely candidates for the natural stimulation required to produce liquefaction. The Rayleigh-wave component of surface waves, (see Bolt 1993) is an obvious source of such natural elongation of the soil column. The fact that surface waves are of much longer period than body waves would seem, on the basis of laboratory testing, to be inconsequential in triggering liquefaction. The Love-wave, the second component of surface waves, causes transverse shear motion propagating in the direction away from the epicentre, and is therefore a likely complementary source of distress to vulnerable deposits. The surface wave strength decreases with depth while the soil consistency, especially if one were to question the prevailing notion that increasing confinement is a negative influence on stability, normally tends to improve. If this were so, at some level within the deposit the diminished power of the surface waves would encounter a soil horizon whose structure was too competent to destroy, at which depth liquefaction could progress no further. This suggests that there is a finite depth at which liquefaction could be brought about by natural forces. So there would appear to be a significant issue involved in deciding which is the real, or operative, triggering mechanism. Is it body waves or surface waves?

The third question, of how the interstitial water tries to accommodate the structural collapse during an earthquake, is now examined to see if this can clarify the understanding. Both potential liquefaction triggering mechanisms will be examined.

The phenomenon of stoping was suggested above as a viable mechanism for the propagation of liquefaction triggered by Shear-waves. Stoping can be viewed by filling a transparent cylinder with a heavy sand slurry so that a loose structure is formed, and then, after the mass consolidates, tapping the side wall at some depth below the sand surface. Collapse of the sand structure occurs beside the impact, and grains settle into a denser packing at that level. The supernatant water becomes apparent as a slug (or pocket of water) which slowly moves upwards through the sand column. During its ascent the slug grows in height as it harvests water from the sand through which it is rising. What is happening is that sand from the loose mass above the slug falls through the water pocket and then, because of having been sedimented by pluviation, accumulates on the surface below the slug in a denser packing. The external impact thus sets up a chain reaction which, progressively working its way steadily upwards through the column, changes a mass of saturated sand from being a disorganized loose (cardhouse) structure into a mass in which grains are oriented long axes horizontally.

Some insight into the alternate liquefaction triggering mechanism was gained by viewing video records from wave flume testing in the Hydraulics Laboratory at the National Research Centre in Ottawa (Becker et al 1986). In the tests, where sandfills were built underwater by the turbulent discharge of a thick slurry, mass liquefaction was triggered by passing water waves over the submerged fill. The onset of liquefaction occurred as a jolt, indicating a simultaneous and instantaneous crisis throughout the full affected depth. Immediately after the crisis, the surface began to settle and to mirror-image the waves, that is, depressions of grade were coincident with water crests. Surface motions became more responsive with the passage of the first few waves after the jolt, but the amplitude started to decline again as the fill regained its stiffness. Stiffness progressed from the bottom upwards as could be seen from the upstream/downstream swaying of marker grains. Once the sand structure had reasserted itself only normal surface erosion resulted from the ongoing passage of waves over the fill. These observations indicate that energy of the type associated with surface waves causes liquefaction within a limited depth all at once. Because $\boldsymbol{k}$ of sand is insufficient to accommodate flow under the prevailing $\boldsymbol{i}_{MAX}$, the mass goes into temporary suspension. Dissipation of ***epwp*** proceeds in pace with re-deposition, as depicted schematically in Figure 15. A slug does not materialize; consequently, there is little room for particle pluviation.

This line of reasoning raises the issue of what the post-liquefaction condition of the mass might be, specifically with regard to its vulnerability to future seismic interference. In other words, does the fact that it previously liquefied make it more or less prone towards future liquefaction? The two separate mechanisms outlined above suggest that the answer would depend on whether collapse had been triggered at depth by body waves, or if surface waves had been the cause. In the former case, the idea that Shear-waves would permit individual particles to form a new structure with predominantly horizontally oriented particles as a result of pluviation through the water slug would argue in favour of a significant improvement. In the case of collapse due to Surface-wave activity, the opportunity for pluviation does not appear likely; therefore, randomly or vertically oriented particles cannot be ruled out. The fact that in the wave flume the collapsed mass eventually attained a packing which was immune to the continued wave train suggests that the mass was rendered proof against liquefaction, at least for that level of wave loading. But, as discussed below, this favourable indication cannot be safely transposed to earthquake induced liquefaction.

In the wave-flume, it was apparent that the strong post-liquefaction performance could well have been dependent, to some extent at any rate, on closer packing brought about by particle translation (swaying) induced by the continuing wave train. If this were so, the question arises as to what would be the final condition of the mass if liquefaction was caused by seismic surface waves in the event the wave train did not persist long enough for all the supernatant water to escape from the surface. In other words, how would the particles be arranged if cyclic lateral translation stopped while the upper part of the deposit was still in suspension? The worst case scenario would be to assume that the particles in the suspended state, if they had room to rotate, would be oriented long-axes vertically, an orientation which would maximize ***k*** and minimize drag in the upward direction. In the absence of a continuing lateral wave stimulus it is difficult to imagine how the particles could recline into the more stable horizontal configuration. It can only be supposed, therefore, that if liquefaction was triggered by a short sequence of Surface-waves, the mass might well be improved towards the base of the affected deposit, but it is conceivable that towards the top, the deposit might well be more vulnerable.

Shear-waves, at least according the conjecture outlined above, have a better post-trauma prognosis than Surface-waves; consequently, an attempt will be made to see if there is any basis, consistent with the hypothesis, which would condone embracing the pervasive belief that liquefaction is the result of Shear-waves. For this purpose two aspects of the mechanism are briefly examined here.

First, the question of whether Shear-waves have the capacity to cause initial structural collapse will be considered. Theoretically, any minor perturbation is sufficient to topple a very loose array, but in practice, liquefaction is associated with powerful events. On the face of it, it appears reasonable to suppose that the strong motions associated with significant earthquakes would provide the required jarring action in the form of inertial force. In this regard it helps to recall that there are many cases on record, in one instance ~~significant earthquakes would provide the required jarring action in the form of inertial force.~~ witnessed by such a credible authority as Evert Hoek, where no perceptible movement was noticed underground during earthquakes which were powerfully felt, and destructive, at ground level. Less persuasive, but nevertheless consistent, are the very low rates of angular shear strain computed by Taylor et al (1985) in deep deposits during strong motions. For instance, in the case of actual earthquake records scaled to a peak horizontal accelerations of 0.28 g, the rate was only about 0.03 degrees per second. Such a gradual swaying of the deposit makes it more difficult to feel confident that collapse would be inevitable, even under high levels of body wave excitation.

Second, the question of whether Shear-waves are capable of generating significant ***epwp*** is considered. For this purpose the seismograph records of 17 Californian standard design earthquakes were searched to determine the velocities involved in such events. It was found that in the worst case the horizontal velocity peaked at 37 cm/s. This was measured during the Imperial Valley May 18, 1940 event for the S90W component recorded at El Centro. As a more recent comparison, this figure was 21 cm/s for the Santa Cruz Mountains October 17, 1989 earthquake for the 0° component recorded at Loma Prieta. Now, if it is assumed that the vibration is carried by the soil skeleton, and that the pore water is completely out-of-phase with the soil skeleton, these relative velocities would be equivalent to V_T or 3.3 mm and 1.3 mm particles, respectively. In other words, the maximum level of ***epwp*** could be generated throughout the mass for all particles up to gravel size. The position taken here is that this situation does not arise since the water cannot move relative to the soil skeleton. This is because of the regional nature of the event there would be scarcely any variation laterally in the deposit, and even if full ***epwp*** were to exist, there would be no gradient to support water movement. Furthermore, and also because of the gradual variation of the seismic wave in the horizontal plane, there would be no source or sink to permit water flow.

On both counts there seems to be no certainty that the more optimistic alternative of Shear-wave triggering can be safely assumed.

LIQUEFACTION FLOW

In practical terms, the power of seepage flow to move grains laterally is limited to the specific instance of discrete particles which are not carrying any burden from overlying grains. Even then, for example, the gradient needed to move an unburdened particle is $i=0.56$ for $\phi_{SS}=33^{\circ}$. Since natural stable horizontal gradients are of the order of 0.1, the best, and perhaps only, opportunity to bring about lateral translation of the mass is immediately following collapse of a liquefiable structure, after the mass has been reduced to a suspension, and before re-deposition returns the particles to a viable structure again.

This suggests that, in the interests of clarity, a distinction be made between the terms "liquefaction" and "liquefaction flow", where the former refers to the condition which follows collapse of a loose structure, and the latter refers to subsequent movement of the temporary suspension. Accordingly, the position taken here is that *liquefaction* is a predominantly vertical movement, and in order for the mass to translate laterally in the form, or as an instance of, *liquefaction flow*, it is necessary that a lateral component of a hydraulic gradient, pre-existing at the site, subsequently act upon it. Liquefaction flow is therefore seen here to be a related, but separate, event to liquefaction.

Under the influence of the external lateral component of gradient it is perhaps possible that particles might change from the vertical collapse orientation to a more horizontal attitude which would facilitate translation, and at the same time, increase the water buffer between themselves. Since the suspension would be in the process of re-deposition during this episode, the base of the stratum would move the least, while motion of the top of the liquefied zone would continue until terminated in accordance with the mechanism, and in the period, sketched in Figure 15.

WATER VENTING

Partially saturated soils in the vadose zone are usually totally impervious to water flow within the time frame of an earthquake, and this holds true for sands as well as finer materials. This is due to the presence of menisci between soil grains in the capillary zone above the water table. The size of the menisci vary with suction head, and at some level above the phreatic surface, conditions will exist such that the menisci fully encapsulate the air in the voids, forming an airlock. At that level, neither can the air get out of the voids, nor can the water get in. This condition persists until the water can take the air into solution.

After a saturated mass below the water table collapses into liquefaction the supernatant water from the re-depositing suspension will accumulate at the base of any overlying impervious boundary, in a manner analogous to water below the loading cap in an undrained triaxial specimen. This water carries the overburden pressure and needs to vent itself to hydrostatic conditions. It will take the easiest route available, and this will be at localized points of weakness in the impervious layer, such as cracks, or interfaces with imbedded structures. High water velocities will occur at these points as the entrapped water from the surrounding region exhausts in a concentrated flow. At these isolated points water velocities can be great enough to cause soil erosion, or sustain quick conditions and sand boiling.

It should be noted that while the entrapped water may influence the stability of the unsaturated overburden, it does not influence any lateral component of hydraulic gradient which may be driving liquefaction flow, nor can it prolong the duration of the soil liquefaction state. So, sand boils and water eruptions are merely evidence of liquefaction having taken place; they are the aftermath of liquefaction, rather than a display of the natural forces as they are actually affecting the mass.