Liquefaction of Saturated Granular Soils

BY

(Research Scientist, Division of Applied Geomechanics, C.S.I.R.O.)

SUMMARY. - The current status of published knowledge on the process of liquefaction in saturated granular soils is reviewed. Methods of determining material properties and initial conditions defining liquefaction response are investigated and their relevance to predictive methods examined. The importance of documenting observed cases of liquefaction under earthquake or other dynamic loading is stressed as being the only means of testing the validity of the investigation and prediction techniques.

I. - INTRODUCTION

Liquefaction is the phenomenon of the partial or total loss of strength and stability of saturated granular soils under the effect of dynamic loading. The source of the dynamic loading may be seismic events, blasting in mines and quarries, or machine foundations, and liquefaction can occur in filled land, embankments, dams, under building foundations and machine bases, and in hydraulically placed mine fill.

The shear strength $\gamma_f$ of cohesionless granular soils under undrained conditions may be expressed as:

$$\gamma_f = (\sigma - \mu_w) \tan \phi'$$  \hspace{1cm} (1)

where $\sigma$ = total normal stress on the plane of failure, $\mu_w$ = pore water pressure, static plus dynamic, $\phi'$ = effective angle of internal friction.

When such a soil is subjected to a suitably severe disturbance the pore water pressure $\mu_w$ may rise to a value equal to the total normal stress $\sigma$, and the shear strength $\gamma_f$ would drop to zero. In this condition the soil exhibits the properties of a viscous liquid and the phenomenon is known as liquefaction.

The process may be described qualitatively in the following sequence.

(i) Seismic energy from blasting or earthquake activity transmitted through the saturated soil mass causes a breakdown in the particle structure.
(ii) The structural breakdown, either sudden or progressive, accompanied by a re-orientation of the soil particles into a denser configuration, increases pore water pressures and reduces shear strength. This process results in some surface settlement of the soil mass.
(iii) When excess pore water pressure becomes equal to the confining pressure, the soil particles go into suspension and the soil mass liquefies. Superimposed structures begin to sink to attain hydrostatic equilibrium.
(iv) Dissipation of the excess pore water pressure is accompanied by a consolidation type process resulting in further surface settlement of the soil mass.

(v) The upward transport of water may maintain the liquefied state at the surface of the soil mass for a considerable period after the cessation of the seismic disturbance.

In consequence, structures which are founded on liquefying material settle as a result of recompaction and consolidation of the soil mass, and as a result of the failure and displacement of the liquefied material. Non-uniform structural and loading conditions, or non-homogeneous soil conditions, may modify the settlement and produce differential settlements or severe tilting of the structures (Ref. 1).

Buried structures (empty underground tanks, pipes and timber piles) experience severe hydrostatic upthrust and may rise to the surface during the liquefied condition (Ref. 2, 1). Embankments or slopes constructed of material that liquefies would spread and slump excessively, or fail by flowing downhill. The surface may exhibit large cracks as a result of lateral movement with corresponding distress to any superimposed structures (Ref. 3).

The dissipation of the excess pore water pressure, combined with general surface settlement, results in surface flooding (Ref. 2). The low permeability of silts and fine sands prevents rapid dissipation of the excess pore water pressure, and surface flooding, together with settlement of superimposed structures, may continue over a period of several minutes after the seismic shaking. If the soil deposits are non-homogeneous in permeability, the dissipation of excess pore water pressures may be concentrated at points of high permeability causing sand blowouts and sand volcanoes to eject considerable quantities of material (Ref. 4).

II. - RECENT EXAMPLES OF LIQUEFACTION

Damage to structures due to foundation material failure, and the collapse of slopes and embankments attributed to liquefaction, have been observed throughout history (Ref. 5). However, it is only recently that the available examples of liquefaction failure have received more than a general, dramatic description. Documented recent cases of widespread liquefaction failure resulted from seismic events in Chile 1960,
TABLE I
RECENT EXAMPLES OF LIQUEFACTION

<table>
<thead>
<tr>
<th>Location</th>
<th>Chile</th>
<th>Alaska</th>
<th>Japan (Niigata)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date</td>
<td>1960</td>
<td>1964</td>
<td>1964</td>
</tr>
<tr>
<td>Earthquake Magnitude</td>
<td>8.4</td>
<td>8.3</td>
<td>7.3</td>
</tr>
<tr>
<td>Maximum Modified Mercalli Intensities</td>
<td>XI</td>
<td>-</td>
<td>VIII</td>
</tr>
<tr>
<td>Epicentral Distances</td>
<td>160 - 400 km</td>
<td>60 - 150 km</td>
<td>60 km</td>
</tr>
<tr>
<td>Soil Conditions</td>
<td>Fluvial and glacial gravels, sands, silts, unconsolidated soil placed by dumping or hydraulically.</td>
<td>Sand layers and lenses in clay deposits; silts, sands and gravels.</td>
<td>Alluvial sands, filled land.</td>
</tr>
<tr>
<td>Phenomena Observed</td>
<td>Settlements and tilting of buildings and foundations, slumping of highway and railway embankments, overturning of bridge abutments and old walls. Mud pumping.</td>
<td>Coastal slope failures, slumping and cracking of beach and deltaic deposits.</td>
<td>Settlement and tilting of buildings, slumping and cracking of surface, slumping of embankments and sliding of slopes, surface inundation and sand boils.</td>
</tr>
<tr>
<td>References</td>
<td>Ref. 5, 3</td>
<td>Ref. 3</td>
<td>Ref. 6, 2</td>
</tr>
</tbody>
</table>

Alaska 1964, and Japan (Niigata) 1964 (Ref. 5, 3, 2, 6). A summary of data is given in Table I. In these cases severe damage to structures was caused by ground failure due to liquefaction rather than the direct effect of vibrations.

Examples of observed liquefaction response have defined areas susceptible to this type of failure. These areas include thick deposits of loose alluvial soils, saturated sand layers and lenses within otherwise stable soil masses, reclaimed land formed by hydraulic filling or dumping without compaction, and slopes and embankments of loose granular material. The problem is to predict the behaviour of any particular mass of granular soil when subjected to a dynamic disturbance. The duration of the disturbance can be short as from blasting, intermediate as from earthquakes, or continuous as caused by traffic or machine foundations. The volume of the soil mass involved is also dependent on the nature of the dynamic disturbance.

III.- REVIEW OF LIQUEFACTION RESEARCH

Liquefaction research has only developed during the last decade and has been directed primarily toward defining the soil properties and ambient conditions that control liquefaction response, and toward quantifying these parameters. Laboratory work has been restricted to the cyclic load testing of small, remoulded samples of soil in specially adapted triaxial loading apparatus or simple shear apparatus, and to the testing of bulk samples of remoulded soil subjected to cyclic or transient accelerations on a shaking table. The laboratory tests of liquefaction response are summarized in Table II.

The use of blasting methods in the field testing of granular soils to determine liquefaction response in situ has been pursued to a limited extent for evaluating limiting values of particle velocity to cause liquefaction (Ref. 7) and extending the range of laboratory results (Ref. 8).

Early research into liquefaction response pursued the concept of a critical acceleration. MOCANU et al. (Ref. 9) applied the term liquefaction to the loss of strength in dry sand subjected to vertical acceleration. The shear strength was found to decrease with increasing vertical acceleration, reaching a value of approximately 20 per cent of the static shear strength at a vertical acceleration of 1 g, thereafter decreasing slowly with further increase in vertical acceleration. The phenomenon was explained by KOLBUSZEWSKI et al. (Ref. 10) in terms of a decrease in effective stress on the potential horizontal failure plane with increasing vertical acceleration. Theoretically, the vertical effective stress would be zero at a vertical acceleration of 1 g (the critical acceleration), with the strength dropping to zero. This condition occurs only over a small part of each cycle of acceleration which, together with the interlocking of the grains, provides some residual strength at accelerations greater than 1 g. The mechanism of reducing the effective stress in this case is not dependent on the development of excess pore fluid pressure, so that neither the number of cycles of loading nor the frequency were controlling factors.

MASLOV (Ref. 11) considered the stability of submerged sandy foundations and structures under the effect of dynamic loading and developed a theory describing the total or partial loss of strength of the sand in such conditions. In the critical acceleration concept proposed, any phenomenon of a dynamic character was completely ruled out at accelerations below the critical value. The critical acceleration for any sand was determined experimentally and was found to depend on the properties of the sand (unspecified), its density (initial relative), the amplitude and frequency of acceleration, and the normal pressure imposed by the overburden and superimposed loads. The critical acceleration was therefore not uniform for the whole sand mass, but varied with depth.
<table>
<thead>
<tr>
<th>Investigator</th>
<th>Year</th>
<th>Sample Type</th>
<th>Sample Size</th>
<th>Type of loading simulated</th>
<th>Test Variables</th>
<th>Parameters Measured</th>
<th>Ref. No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>MASLOV</td>
<td>1957</td>
<td>Bulk Sample</td>
<td>Up to 25 ton</td>
<td>Vertical &amp; Horizontal Vibrations</td>
<td>Porosity, Acceleration Type of sand</td>
<td>Pore water pressure</td>
<td>11</td>
</tr>
<tr>
<td>FLORIN</td>
<td>1961</td>
<td>Bulk Sample</td>
<td>20cm thick</td>
<td>Impact and Vibration</td>
<td>Porosity</td>
<td>Pore water pressure, Time of liquefaction</td>
<td>12</td>
</tr>
<tr>
<td>HUANG</td>
<td>1961</td>
<td>Triaxial</td>
<td></td>
<td>Vertical Vibration</td>
<td>Void ratio, Confining pressure Acceleration</td>
<td>Pore water pressure</td>
<td>13</td>
</tr>
<tr>
<td>SEED</td>
<td>1966</td>
<td>Triaxial</td>
<td></td>
<td>Shear Waves</td>
<td>Void ratio, Confining pressure Cyclic stress amplitude Cyclic strain amplitude</td>
<td>Axial strain, Pore water pressure Cyclic stress Number of cycles</td>
<td>15</td>
</tr>
<tr>
<td>LEE</td>
<td>1967</td>
<td>Triaxial</td>
<td></td>
<td>Shear Waves</td>
<td>Void ratio, Confining pressure Cyclic stress amplitude Failure criterion</td>
<td>Axial strain, Pore water pressure Number of cycles Cyclic stress</td>
<td>16</td>
</tr>
<tr>
<td>SCHOEDER</td>
<td>1968</td>
<td>Triaxial</td>
<td></td>
<td>Compressional Waves</td>
<td>Void ratio, Effective stress ratio Continuing pressure Initial pore pressure Cyclic stress amplitude</td>
<td>Axial deformation, Pore water pressure Number of cycles</td>
<td>17</td>
</tr>
<tr>
<td>YOSHIMI</td>
<td>1967</td>
<td>Bulk Sample</td>
<td>50x25x27cm</td>
<td>Horizontal Vibration</td>
<td>Void ratio, Surcharge pressure Acceleration Frequency</td>
<td>Pore water pressure, Acceleration</td>
<td>19</td>
</tr>
<tr>
<td>TANIMOTO</td>
<td>1968</td>
<td>Bulk Sample</td>
<td>80x80x130cm</td>
<td>Horizontal Impact</td>
<td>Void ratio</td>
<td>Pore water pressure, Acceleration Surface settlements</td>
<td>20</td>
</tr>
<tr>
<td>PEACOCK</td>
<td>1968</td>
<td>Simple Shear</td>
<td>6x6x2cm</td>
<td>Shear Waves</td>
<td>Void ratio, Confining pressure Cyclic stress amplitude Frequency</td>
<td>Shear strain, Pore water pressure Shear stress Number of cycles</td>
<td>21</td>
</tr>
<tr>
<td>PRAKASH</td>
<td>1970</td>
<td>Bulk Sample</td>
<td>105x60x25cm</td>
<td>Horizontal Vibration</td>
<td>Type of sand, Void ratio Acceleration Surcharge pressure</td>
<td>Pore water pressure, Number of cycles Surface settlements</td>
<td>8</td>
</tr>
<tr>
<td>FINN</td>
<td>1970</td>
<td>Triaxial</td>
<td></td>
<td>Shear Waves</td>
<td>Void ratio, Confining pressure Cyclic stress amplitude Strain history</td>
<td>Axial strain, Shear strain Pore water pressure Number of cycles</td>
<td>23</td>
</tr>
</tbody>
</table>

Theoretical expressions were apparently adequately confirmed by experiment using small sample laboratory methods and large capacity (9 and 25 ton) field equipment.

FLORIN et al. (Ref. 12) defined the liquefaction conditions as the collapse of the structure of the sand, with the possibility of sand consolidation, and either partial or total saturation of the sand with water. The criteria of collapse proposed were intensity of disturbance, stress condition and weight of surcharge, and hydraulic gradient of water flow. The displacement of the soil surface and superimposed structures was then determined by the duration of the liquefied condition and the viscosity of the liquefied mass. This duration was found to depend on the thickness of the layer, the permeability, rate of change of volume of voids, intensity and location of drainage and duration of dynamic loading.

The change in porosity was observed experimentally
by changes in conductivity, pure pressure measurement using membrane type cells; the change in viscosity by the sinking of heavy bodies placed on the surface.

FLORIN found that under impact loading, a large sample of loose, saturated sand liquefied simultaneously over the entire depth of the responding stratum, and that increase of impulse intensity led to an increase in the depth of the liquefied zone, but not to an increase in the sand consolidation. The increased depth of the responding stratum appears to be consistent with MASLOV's critical acceleration concept. Consolidation took place from the bottom of the liquefied zone, with the surface layers remaining in the liquefied condition. Subsequent impact loads of the same intensity produced decreased depths of liquefaction.

Continuous vibration of a large sample produced liquefaction from the surface downwards. The surface layers liquefied first because of low overburden pressure, which in turn reduced the overburden pressure on the underlying layers and permitted them to liquefy. This description of the sequence of events masked the effect of the increase number of cycles of loading before liquefaction occurred in the lower layers. As soon as the entire stratum had liquefied consolidation occurred from the bottom up.

Based on laboratory experience, FLORIN concluded that all sufficiently loose, granular soils of any grain size may liquefy, as liquefaction is only an intermediate state between two stable states of compaction. The time, however, during which the liquefied state persists may vary greatly; for example, after the first impact of an impact test, a coarse sand remained liquefied for approximately 2 sec, while a fine sand remained liquefied for approximately 30 sec.

The presence of overburden required an increase in the intensity of the disturbance. FLORIN concluded, therefore, that at depths below the ground surface in excess of 10 to 15 m even very loose sands cannot be liquefied.

Using as an empirical indicator the surface settlement (in excess of 8 cm) at a radius of 5 m, FLORIN also proposed a 'standard' grain blasting method (5 kg of explosives at 8-10 m deep) to determine susceptibility to liquefaction in the field.

HUANG (Ref. 13) investigated the liquefaction resistance of a remoulded silty sand sample under triaxial conditions as opposed to the anisotropic confining conditions experienced in bulk samples. The triaxial sample, with various values of vertical and lateral stress imposed, was subjected to vertical acceleration on a shaking table. The results showed that the developed pore water pressure decreased as the initial density, lateral and vertical stress increased, and acceleration decreased.

HUANG suggested that, using the results of the tests described, it was possible to determine the pore water pressure developed at any point in a given sand foundation under any dynamic loading. The key to the solution was to find the relationship between developed pore water pressure, intensity of dynamic action and state of stress in the sand mass. At that stage, the duration of the dynamic loading, implying progressive build-up of pore water pressure, was still not considered to be an important parameter.

Using dimensional analysis, BAZANT (Ref. 14) developed a concept of liquefaction acceleration which defines a critical value below which liquefaction cannot occur. The liquefaction acceleration was found to be a function of the soil density, the frequency of the vibration, the depth of the layer and the amplitude of the ground motion. The concept was developed from a careful derivation of dynamic stability of saturated sand. Stability was defined as the occurrence of non-occurrence of compaction under the action of a vertical acceleration. BAZANT assumed that earthquakes imparted predominantly vertical acceleration in the zones of interest; that is, longitudinal waves propagating vertically from the bedrock through the soil layers causing alternating compression and dilation within the soil. The presence of transverse waves was assumed to assist the compaction and increase the possibility of liquefaction and the loss of stability. BAZANT concluded that the measurement of profiles of soil density is one of the crucial points of examination in the investigation of the dynamic stability of an area.

The experimental studies of SEED et al. (Ref. 15) were the first to move away from the concept of critical acceleration and attempted to simulate the performance of soil elements within a soil mass subjected to an assumed seismic loading. The critical acceleration values, developed theoretically and experimentally previously to this time, were considered by SEED to be influenced by the frequency and duration of the shaking before liquefaction occurred and possibly also by the geometry and deformation of the container.

An element of soil within a soil mass is subjected to a complex system of loading, but SEED assumed that the major part of the soil deformation could be attributed to the upward propagation of shear waves from underlying layers. Therefore, each element of soil may be considered as being subjected to a series of cyclic shear stresses and strains. This condition, as imposed on a soil element under a level ground surface, could be approximately reproduced on a triaxial sample when tested in cyclic compression under undrained conditions.

Using this method, SEED established that under cyclic loading the pore water pressure gradually increased until it equalled the initial confining pressure. The deformation then increased suddenly and the soil was said to have liquefied. The number of cycles of loading required to cause liquefaction were found to depend on the initial void ratio, the initial confining pressure and the magnitude of cyclic stress. The authors concluded that the danger of liquefaction of a saturated sand as a result of cyclic loading was determined qualitatively by the following:

(i) The void ratio of the sand - the higher the void ratio the more easily liquefaction will occur.
(ii) The confining pressure acting on the sand - the lower the confining pressure the more easily liquefaction will develop.
(iii) The magnitude of the cyclic stress or strain - the larger the stress or strain the lower the number of cycles required to induce liquefaction.
(iv) The number of stress cycles to which the sand is subjected.

LEE et al. (Ref. 16) also used triaxial samples subjected to cyclic loading to quantify the qualitative
relationships established by SEED et al. (Ref. 15). LEE defined and used a number of failure criteria to study the susceptibility of sand to liquefaction and the development of large shear strains. The authors concluded that liquefaction resistance was determined by a complex, as yet undefined, relationship between the factors considered. However, over some ranges of the variables, approximately linear relationships existed, for example, between the relative density and the cyclic stress, and the confining pressure and the cyclic stress required to cause initial liquefaction in a given number of cycles of loading. These approximate relationships allow an estimate of the depth at which initial liquefaction takes place under any variation of cyclic stress with depth within the soil.

SCHROEDER et al. (Ref. 17, 18) also used a triaxial apparatus to investigate the liquefaction potential of sand samples under various conditions. Disregarding the effect of S waves (shear), SCHROEDER assumed that the primary cause of liquefaction was due to the arrival of P waves (longitudinal) at the site of interest, and tested the triaxial samples accordingly. The sudden onset of liquefaction was not observed, but a gradually increasing axial strain with increase in the number of cycles of loading was observed. The developed pore water pressures appeared to be asymptotic to a peak value which increased with increasing void ratio. It was found that the pore water pressure increased and hence liquefaction tendency increased with:

(i) decreasing initial effective stress ratios,
(ii) decreasing initial effective confining pressures,
(iii) increasing disturbance, and
(iv) increasing ratio of initial pore water pressure to initial intergranular pressure.

These findings were in general agreement with those of SEED et al. (Ref. 15) but the tests did not reproduce sudden liquefaction.

YOSHIMI (Ref. 19) conducted tests on large samples in a rigid box covered with an impervious membrane, with surcharge applied, and subjected to horizontal vibrations. The difference between this and other bulk sample tests (Ref. 12) was the presence of the impervious membrane and the disregard for the critical acceleration concept. YOSHIMI observed the pore water pressure increase at various depths within the sample (50 x 25 x 27 cm) and discovered that the process leading to complete liquefaction consisted of two distinct steps. The first was a period during which the pore water pressure at different depths increased uniformly and simultaneously while the sand remained stable, and the second was a very rapid rise in pore water pressure until it reached a value approximately equal to the total stress at that particular depth.

YOSHIMI idealized the pore water pressure-time curve into five distinct stages:

(i) Gradual increase in excess pore water pressure due to initial compaction process.
(ii) Sharp increase in pore pressure to a peak value, with sudden liquefaction.
(iii) Liquefied state with no change in pore water pressure.
(iv) Consolidation process proceeding from the bottom up with gradual decrease in pore water pressure.

(v) Stabilized state with excess pore water pressure at level of surcharge pressure in the constant volume apparatus.

These observations generally confirmed those of SEED et al. (Ref. 15), obtained by using the triaxial apparatus with considerably different boundary conditions and method of loading.

TANIMOTO (Ref. 20, 21) also observed distinct stages of development in the liquefaction process but in terms of surface settlement as well as excess pore water pressure development. TANIMOTO used an 80 cm square sand bin, 150 cm high, mounted on a vibration table and subjected to horizontal impact loading. The total surface settlement due to a single impact was found to consist of two components, one attributed to the temporary collapse of the granular structure of the sample leading to the development of excess pore water pressure, and the other to the dissipation of the excess pore water pressure accompanied by the squeezing out of the pore water. The latter phenomenon appeared to approximate general consolidation process. The magnitude of the two displacements appeared to be approximately equal, while the response times were in the approximate ratio of 1:100.

PEACOCK et al. (Ref. 22) developed and used a new sample testing technique which simulated the assumed ground conditions rather better than the triaxial test method, and it was claimed that this was the best possible means of evaluating quantitatively the stresses inducing liquefaction. The method applied a cyclic horizontal shear load to a small prismatic sample (6 cm x 6 cm x 2 cm deep) of remoulded sand which was subjected to a vertical surcharge load. The general relationships between initial relative density, peak-pulsating shear stress, confining pressure and number of cycles of loading to produce liquefaction, were found to be substantially the same as those observed with triaxial samples. However, the value of the peak-pulsating shear stress to cause liquefaction in a given number of cycles in the simple shear apparatus was found to be approximately one third of that in the triaxial apparatus. The discrepancy could be partly attributed to the difficulty in preparing uniform samples in a sharp-cornered mould. Samples of non-uniform initial relative density appeared to have increased susceptibility to liquefaction. Special preparation of samples indicated that the value of the peak cyclic shear stress, required to cause initial liquefaction under specified conditions, increased to approximately half of that in the triaxial tests. The discrepancy may also be attributed to differences in confining conditions found in the triaxial sample and the simple shear sample.

PRAKASH et al. (Ref. 8) used a vibration table and bulk sample to determine the liquefaction response of different sands. The sample size was 105 cm x 60 cm x 25 cm deep and a vertical surcharge could be applied. The general relationships between relative density, confining pressure, acceleration, and duration of loading were much the same as noted by other workers. One finding of interest was that, under an initial surcharge corresponding to an overburden of about 4 m, the excess pore water pressure developed was negligible, and indicated that liquefaction may not occur below this depth in the sand tested.

FINN et al. (Ref. 23) extended the methods of triaxial and simple shear testing used by SEED et al. (Ref. 15) and PEACOCK et al. (Ref. 22) and developed a
procedure for comparing the results of simple shear and triaxial tests to show that they are equivalent and consistent with each other. The liquefaction resistance (number of cycles of loading to cause liquefaction) of a particular sand is uniquely defined by the initial effective confining pressure, the peak cyclic shear stress and the initial void ratio if the remoulded sand sample has not undergone any previous shear strain. For all other sand samples the liquefaction resistance appears to be predominantly controlled by the strain history of the sand.

FINN has indicated that a threshold value of the previous shear strain exists for a particular sand below which liquefaction resistance is increased, but above which liquefaction resistance is drastically decreased. All other controllable conditions are substantially identical. This change in liquefaction resistance has been attributed to the formation of a new soil structure which is particularly weak in resisting cyclic shear loads. The threshold value also depends on the number of times the particular shear strain has been cycled.

This work throws doubt on the validity of predictive techniques utilizing liquefaction response relationships observed on remoulded soil samples, and indicates the need to develop methods of in situ testing where the effects of the previous strain history of a soil mass are preserved. In any field situation, however, great difficulties are encountered in determining the material properties.

IV. PREDICTION OF LIQUEFACTION RESPONSE

Research into liquefaction response has established that the resistance to liquefaction, expressed in the number of cycles of loading is defined by the confining pressure, the relative density, the magnitude of the cyclic stress and the strain history. No functional relationships linking the parameters involved, have been developed, and therefore no analytical solutions are available to predict the performance of a soil mass prone to liquefaction. However, various methods of conceptual, mathematical and physical modelling are available which can give an indication of the response under assumed loading conditions. In many cases the prediction may be restricted to defining whether liquefaction will or will not occur without indicating the extent or the duration.

For example the response of a soil mass with a horizontal surface may be investigated, and the depth at which initial liquefaction will occur predicted, using the results of laboratory investigations (Ref. 16). The confining stress in such a case increases linearly with depth of overburden. Laboratory tests indicate that the cyclic stress required to cause liquefaction in a specified number of cycles increases approximately linearly with confining pressure. The depth to initial liquefaction may then be defined for any variation of cyclic stress amplitude with depth. Liquefaction would first occur at the depth where the assumed cyclic stress variation corresponds with the least number of cycles required to cause liquefaction at a particular cyclic stress amplitude. Thus, if the assumed cyclic stress amplitude distribution is uniform with depth, liquefaction will be initiated at the surface and progress downwards with increasing number of cycles, assisted by the progressively reducing overburden pressure on the underlying soil as the surface layers liquefy. Alternatively, if the assumed cyclic stress amplitude increases uniformly with depth the whole layer may be expected to liquefy simultaneously after the requisite number of cycles.

To predict response to impact loading, AMBRASEY et al. (Ref. 24) have proposed an alternative relationship by defining a critical particle velocity, relating to the cyclic shear stress above which shear failure occurs. Liquefaction under cyclic loading condition is initiated by progressive shear failure, but this will not take place until the critical particle velocity is exceeded. The relationship includes a pore pressure parameter which depends on the time histori, of the loading, but the form of this dependence has not been defined.

From the above considerations it follows that, initial liquefaction occurs within a soil mass at a point below the surface, the surface layers will be protected from further loading because the liquefied stratum cannot transmit shear stresses.

With structures of complex geometry, such as building foundations, slopes, embankments and retaining walls, it is necessary to modify the previously described method because of the unknown distribution of confining stresses. One method, used in analysing the response of a dam founded on liquefaction-prone soil, has been described by SEED et al. (Ref. 25). The technique uses the finite element method of determining the initial static stress distribution within the soil mass. The base boundary is then subjected to an assumed seismic loading and the time history of dynamic stressing of each soil element is inspected after an appropriate number of loading cycles. Laboratory cyclic loading tests, conducted under simple shear conditions, are made to determine the equivalent uniform cyclic stress required to cause liquefaction in a given number of cycles of loading for a range of initial confining pressures.

After each period of loading, the time histories of the cyclic dynamic shear stresses are equated to an equivalent number of cycles of uniform stresses. These values are compared to the stresses required to cause liquefaction under the appropriate confining stress conditions, the liquefied zone defined, and the stability of the structure investigated. Since the liquefied zone cannot transmit stress, a new 'initial' stress distribution is defined for the next period of loading. The process is repeated until the end of the period of seismicity, and the increase in the extent of the liquefied zone can be followed progressively and the stability of the structure investigated at the end of each arbitrary chosen period of loading.

Another method of predicting the response of a complex structure is to use physical models. KRISHN et al. (Ref. 26, 27) used this method to investigate the possible liquefaction of foundation material under a dam. A scale model (1:200) of the dam and foundations was constructed and subjected to an approximately scaled periodic acceleration. The sand tested was found not to be prone to liquefaction under the anticipated ground motions at a relative density of 20 per cent. However, settlement due to compaction under the assumed ground motion would be of appreciable magnitude.

Field testing of liquefaction-prone material in situ is also a means of predicting whether liquefaction
VI. CONFIRMATION OF PREDICTIONS

In the development of any methods of predicting response, the most difficult step is the field observation of the response of the prototype structure to the design loadings. This step is necessary to determine the adequacy of the prediction technique and the relevance of the material properties and initial conditions used in the prediction. With liquefaction response the problem is compounded by the relatively infrequent occurrence of the design conditions and the difficulty in determining the margin of safety, if the prototype is subjected to the design loading and liquefaction failure has not actually taken place. Consequently, it is necessary to investigate any available examples of liquefaction failure.

The relatively recent development of an awareness of the factors defining liquefaction response has precluded the use of this knowledge in previous investigations of liquefaction-prone areas. While soil conditions after liquefaction may be studied in detail, the relevant soil conditions prior to the event are not usually known nor is the history of seismic shaking.

This problem was encountered by MARSAL (Ref. 29) in describing the effects of the Jaliscan (Mexico) Earthquake of 1959. No detailed information was available of the soil conditions existing prior to the earthquake, and the soil properties defined and tests used after the earthquake were not directly related to liquefaction response. The conclusions that structural damage resulted from soil liquefaction appear to have been drawn from the magnitude and nature of settlements and displacements rather than as a result of the subsequent soil exploration.

Notable exceptions to this problem occur when studying the response of man-made soil structures, such as dams or embankments, where detailed construction records have been kept, and in recently developed areas of cities whose relevant material properties existing prior to the seismic event, may be deduced from the results of conventional site investigation procedures. The latter case occurred in Niigata (Ref. 30, 6) where in the course of development many borings and standard penetration tests had been carried out. It was possible to construct pre-earthquake soil profiles in terms of SPT N-values. As a result of ground failure through liquefaction, the area of the heaviest structural damage corresponded to the area of recent fluvial deposition and was also enclosed by the line defining the area of N = 20 or less at a depth of 5 m.

From the analysis of soil liquefaction at Niigata and other investigations it can be said that a saturated granular soil is prone to liquefaction under the effects of a moderate seismic loading (Modified Mercalli Intensities of VII-VIII), if the standard penetration test value N = 20 or less in the top 10 m. Standard penetration test results are usually available from conventional site investigations and require no specialized techniques to be developed. The test procedure imposes a standard intensity impact loading on the soil in situ. The resulting number of blows per unit displacement recorded depend in some complex manner on the relative density and confining pressure. The dynamic nature of the loading may cause liquefaction of a small volume of soil at the tip of the sampler thereby increasing the penetration per blow and reducing the N-value in liquefaction-prone soils.

In areas subjected to seismic activity, with structures founded on liquefaction-prone soil, it is desirable to investigate and measure the soil properties relevant to liquefaction response so that, if liquefaction occurs, the accuracy of methods of prediction can be confirmed. The soil parameters that require to be known are the relative density, the permeability, grain size distribution, particle orientation and soil fabric, stress-strain relationships, dynamic shear strength, and damping factors. This approach, however, would not be economical, and it may take a very long period of time before an adequate amount of useful information is accumulated.

Two approaches are available to overcome this problem. One method would be to establish a passive observational experiment in an area of known, high seismicity and suitable soil conditions. The soil conditions at the chosen site would be defined in terms relevant to liquefaction response and large concrete blocks of known weight and geometry constructed on the surface of the soil. Seismic accelerations would be monitored using appropriately located strong motion instruments, and the soil response monitored in terms of surface displacements, settlements of the concrete blocks and pore water pressure variations with time and depth. This would provide a means of obtaining relevant field observations over a reasonable period of time. Alternatively, if seismic events at a suitable site are not sufficiently frequent, explosives could be used to provide a seismic disturbance of desired duration and intensity.

VII. SUMMARY AND CONCLUSIONS

Saturated granular soils, when subjected to seismic loading of sufficient duration and intensity, undergo a breakdown of the granular structure with attendant rise in pore water pressure. When the pore water pressure becomes equal to the total confining pressure, the effective intergranular stress becomes zero and the soil mass loses its strength. The soil now behaves as a viscous liquid and the phenomenon is termed liquefaction.

Laboratory methods have been developed that allow the material and environmental parameters controlling liquefaction response to be defined and general relationships to be established. This work has indicated that liquefaction resistance of a soil, expressed in terms of the number of cycles of loading necessary to cause liquefaction, is dependent on the initial relative density, the initial effective confining pressure, the magnitude of the cyclic shear stress, and the strain history of the soil. General trends have been established but no functional relationships are available defining the interrelationship between these parameters.

The results of laboratory investigations may be used to predict the response to seismic loading of liquefaction-prone soils. Prediction would in most cases be restricted to an indication of whether liquefaction would or would not occur, but no indication can be given of the extent of the liquefied zone or
the response of the system after liquefaction. Field
tests using standardized blasting methods may also be
used to predict whether liquefaction will occur under
some assumed seismic loading.

Numerical methods are being developed which allow
the development of the liquefied zone to be followed
and the total stability of the soil mass and super-
posed structures to be investigated progressively.
These methods utilize the results of laboratory tests
on small, remoulded soil samples, and difficulties may
be encountered in predicting the performance of natur-
ally occurring soil deposits where the strain history
is unknown.

The most difficult step in developing a means of
reliable liquefaction response prediction is the con-
firmation of the accuracy of the prediction. Analysis
of observed liquefaction response is not very often
helpful as the soil conditions prior to liquefaction
are rarely available. This emphasizes the need for
specially constructed field observation programs and
the development of techniques of determining soil
properties relevant to liquefaction response in situ
so that the effects of the strain history are not
destroyed.

REFERENCES

1. SEED, H.B. - Soil Stability Problems Caused by
   Earthquakes. Bituminous Materials Research Lab-
   oratory Rep., Dep. of Civil Engng., University of
   California, Berkeley, January 1967.

2. KAWASUMI, H. (Editor-in-Chief). - General Report
   on the Niigata Earthquake of 1964, Tokyo Elec-

3. SEED, H.B. - Landslides During Earthquakes Due to

   Bull. Seismological Society of America, 1958,
   Vol. 48, pp. 155-161.

5. DUKE, C.M. and LEEDS, D.J. - Response of Soils,
   Foundations, and Earth Structures to the Chilean

6. SEED, H.B. and IDRISI, I.M. - Analysis of Soil
   Liquefaction Niigata Earthquake. A.S.C.E. Jour-
   83-108.

7. PUCHKOV, S.V. - Correlation between the Velocity
   of Seismic Oscillations of Particles and the
   Liquefaction Phenomenon of Water-Saturated Sands.
   Problems of Engineering Seismology. Consultant

8. PRKASH, S. and GUPTA, M.K. - Liquefaction and
   Settlement Characteristics of Loose Sands under
   Vibration. Proc. Conf. on Dynamic Waves, Univer-

9. MOGAMI, T. and KUDO, K. - The Behaviour of Soil

10. KOLBUSZESKI, J. and ALYANAK, I. - Effects of
    Vibrations on the Shear Strength and Porosity of
    Sands. The Surveyor and Municipal Engineer, 1964,
    May 30 and June 6.

11. NASLOV, N.N. - Questions of Seismic Stability of
    4th Int. Conf. Soil Mech. and Fdn Engng, 1957,
    pp. 368-372.

12. FLORIN, V.A. and IVANOY, P.L. - Liquefaction of
    Saturated Sandy Soils. Proc. 5th Int. Conf. Soil

13. HUANG, W. - Investigation on Stability of Satura-
    ted Sand Foundations and Slopes against Liquefac-
    tion. Proc. 5th Int. Conf. Soil Mech. and Fdn

14. BAZANT, Z. - Stability of Saturated Sand during
    Earthquakes. Proc. 3rd World Conf. on Earthquake

15. SEED, H.B. and LEE, K.L. - Liquefaction of Satura-
    ted Sands during Cyclic Loading. A.S.C.E. Jour-
    105-134.

16. LEE, K.L. and SEED, H.B. - Cyclic Stress Condi-
    tions Causing Liquefaction of Sand. A.S.C.E.
    Journal of Soil Mech. and Fdn Div., 1967, SME,
    pp. 47-70.

17. SCHROEDER, W.L. and SCHUSTER, R.L. - Liquefac-
    tion Phenomena in Saturated Sands. 7th Annual Proc.
    Symp. on Engng Geol. and Soils Engng, University
    of Idaho, 1968.

18. SCHROEDER, W.L. and SCHUSTER, R.L. - Laboratory
    Simulation of Seismic Activity in Saturated Sands.
    Vibration effects on earthquakes on Soils and

19. YOSHIMI, Y. - An Experimental Study of Liquefac-
    tion of Saturated Sands. Soil and Fdn, The Japan-
    ese Society of Soil Mech. and Fdn Engng, 1967,

20. TANIMOTO, K. - Liquefaction of Sand Layer Subject-
    ed to Shock and Vibratory Loads. Proc. 3rd Asian
    Reg. Conf. on Soil Mech. and Fdn Engng, 1967,

21. TANIMOTO, K. - Study of Settlement Associated with
    Liquefaction of Sand Subjected to Impact Loads.
    Trans. of Japanese Society of Civ. Engrs, 1968,
    No. 152, pp. 39-42.

22. PEACOCK, W.M. and SEED, H.B. - Sand Liquefaction
    under Cyclic Loading, Simple Shear Conditions.
    A.S.C.E. Journal of the Soil Mech. and Fdn Div.,
    1968, SME, pp. 685-708.

23. FINN, W.D.L., BRANSBY, P.L. and PICKERING, D.J.
    - Effect of Strain History on Liquefaction of Sands.
    SME, pp. 1117-1134.

24. AMBRAEY, N. and SARMA, S. - Liquefaction of
    Soils Induced by Earthquakes. Bull. Seismologi-
    cal Society of America, 1969, Vol. 59, No. 2,
    pp. 651-664.


