GEOTECHNICAL EARTHQUAKE ENGINEERING

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The aim of **Geotechnical Earthquake Engineering** is two fold:

1. The study of geotechnical materials such as soils and rocks under seismic loading conditions

2. Analysis and design of earth structures and foundations including soil layers and slopes subjected to earthquakes.

The first part of the course gives an overview of dynamic soil properties including the liquefaction phenomenon. Response analysis of soil layers gives an idea of how the local soil conditions effect the ground motion at a site. Seismic response and stability of earth dams show the simplified design analysis technique. Seismic earth pressures and the seismic design of foundations are also included.

1. DYNAMIC SOIL PROPERTIES

Dynamic response of a soil mass depends on loading conditions. For example, the response for small strain loading is different from that for large strain loading. While for small strain loading, we may treat soil as a visco-elastic material with quite good accuracy, the assumption of visco-elastic material may not be valid for large strain loading. Earthquake loading may be either small strain or large strain loading. Similarly, the behaviour under monotonic loading and cyclic loading may be different. In cyclic loading, the soil strength may degrade. The response may be different depending on the frequency of the loading, particularly for saturated soils. Under seismic loading, for saturated soils, the behaviour is undrained. Also speed of loading has its effect. Also, when sliding occurs (strain is no longer applicable), the strength may depend on the amount of displacement.

Soil has non-linear material properties with limited strength and quite often, the strength degrades under cyclic loading conditions. However, more often than not, we use soil as a visco-elastic material which is therefore a simplification. When we use soil as elastic material for modeling purposes, we should keep in mind the limited strength characteristics.

1.1 Elastic properties

The elastic properties of the soil is given by the mass density, elastic modula and the damping values. The elastic modula are defined by two parameters which may be any of the following combinations.

a) The Lame's constants λ and μ;

b) Youngs Modulus E and Shear Modulus G, ($G=\mu$);

c) E or G and Poissons Ratio ν;

d) Compression Wave Velocity (C or V_p) and Shear Wave Velocity (S or V_s);

e) C or S and the ratio $\alpha = C/S$.

The following relationships may be useful.

$$\lambda = \frac{E\nu}{(1+\nu)(1-2\nu)} \qquad \mu = \frac{E}{2(1+\nu)}$$

$$C = \sqrt{\frac{\lambda + 2\mu}{\rho}} \qquad S = \sqrt{\frac{\mu}{\rho}} \qquad \alpha = \frac{C}{S} = \sqrt{\frac{2(1-\nu)}{(1-2\nu)}}$$

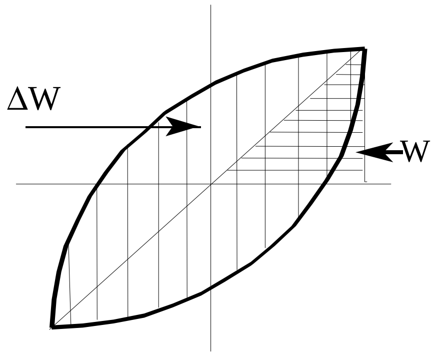
Where ρ is the mass density. Lame's Constant μ and shear Modulus G are one and the same.

The wave velocities combine the elastic modula and the mass densities. In soil, the shear wave velocities (or the shear modulus) and the damping values are the most important parameters. In saturated soils, particularly when under water, when the compression wave velocity is measured in the field, it often appears to be the velocity of sound waves in water.

When a soil sample is taken through a loading-unloading-reloading cycle, the stress-strain graphs show a hysteresis loop. See figure 1. The area under the loop gives the energy that is lost while going through a cycle. This loss of energy is related to the damping factor in a visco-elastic solid. The shear modulus is represented by the secant modulus at any given strain. The damping coefficient is related to the energy loss by the relationship:

$$\lambda = \frac{1}{4\pi}\frac{\Delta W}{W}$$

where W = Area under the loop and
ΔW = Area under the triangle
as shown in the figure 1.
<u>(Do not confuse the damping coefficient with the Lame's Constant. We would not use Lame's Constant again.)</u>



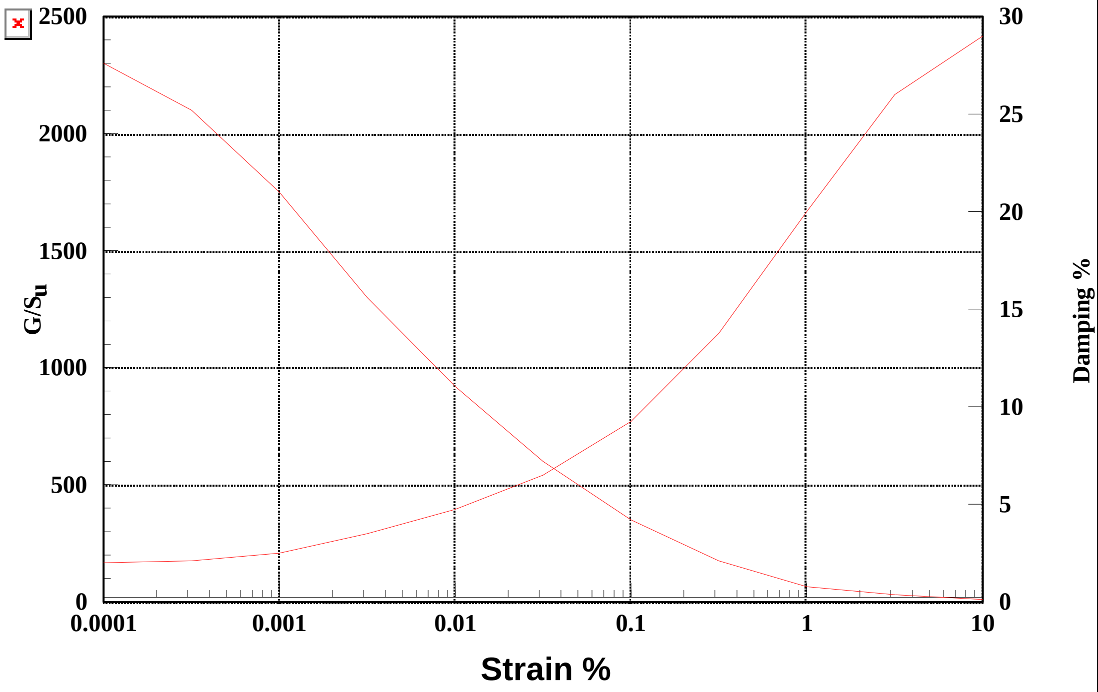
<u>Figure 1</u>

The shear modulus and damping of soil depends on the strain level. Higher the strain level, the lower is the modulus and higher is the damping. Figures 2 and 3 show relationships of the modulus and damping with the strain level, Seed et al (1973). In general, G at any strain γ is related to the G_{max} value at low strain although the relationship differs from soil to soil.

Field measurements of shear wave velocities generally give those at the very low strain level. Soil type, void ratio, initial effective confining pressures, over consolidation ratio, plasticity index are some of the factors which influence the shape of the curves, the value of the G_{max} and the damping factor, Dobry & Vucetic (1987), Vucetic & Dobry (1991). Empirical relationships for G_{max} can be found in the literature, Richart, Hall & Woods(1970), Hardin (1978), Hardin & Drnevich(1972) etc.

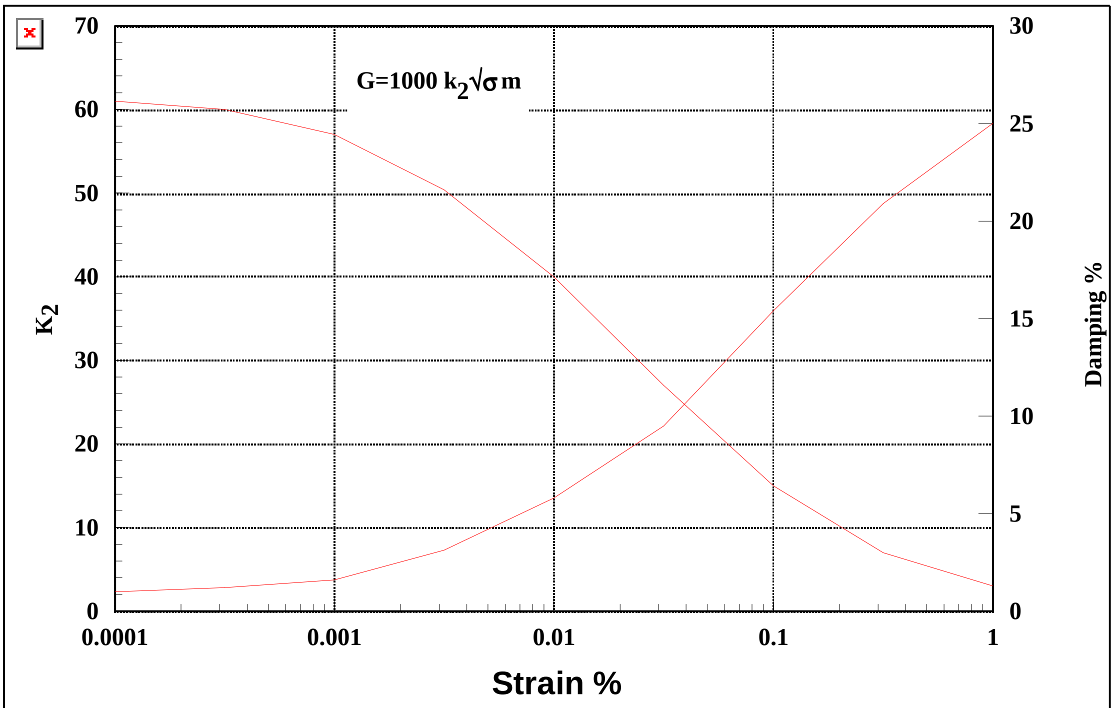
When subjected to undrained cyclic loading, the stiffness and the strength of the soil degrades with number of cycles due to the increase of pore water pressures while the damping increases.

The limit strength of soil under seismic loading conditions may be expressed by the total undrained strength or by the effective strength associated with the pore water pressure. The undrained strength and the pore water pressures are stress path dependent and therefore limit strength of soil depends on how the failure state is arrived at. Since a soil element in the field is subjected to 3 normal and 3 shear stress components, the limit strength will depend on how each component is varied in the field during an earthquake. Therefore, prediction of soil strength under field conditions during earthquake is a very complex problem.



Shear Modulus and Damping of Clay

Figure 2



Shear Modulus and damping of sand

Figure 3

1.2 MEASUREMENT OF SOIL STRENGTH

1.2.1 laboratory testing of soils:

Soil may be tested in the laboratory in the following devices:

a) Cyclic triaxial tests;
b) Cyclic simple shear tests;
c) Resonant column tests;
d) Other specialist tests such as in hollow cylinder apparatus or in ring shear apparatus.

In these tests, only some of the components of the variations of the stress conditions may be applied. Resonant column apparatus is designed to determine the modulus and damping of soils but the limit strength cannot be determined in this test.

The cyclic triaxial and simple shear tests and the specialist tests can be performed for low strain and high strain conditions. The resonant column tests are generally performed at low strains. Triaxial tests can be performed both under drained and undrained conditions with measurements of pore water pressures while in simple shear tests, no pore water pressures are measured. The frequency of loading generally depends on the capability of the measuring devices.

1.2.2 In-situ tests

These tests are generally applied in the seismic survey to determine the wave velocities. The following tests can be used:

a) Seismic reflection and refraction survey;
b) Seismic cross-hole survey;
c) Seismic up or down hole survey.

In all these methods, an explosive charge is detonated at a given point and the arrivals of different kinds of waves are noted by sensors at other points. From the time taken by different waves to arrive at the sensor, average wave velocities are estimated.

Other field tests include:

d) SPT tests
e) CPT tests.

These two tests determine the resistance of soil (i) by the number of blows required for the apparatus to penetrate a given distance (SPT) or (ii) by the force required to push the cone penetrometer at a given rate (CPT) and then these numbers are correlated to other soil properties.

SPT test is performed by using a 140 lb (63.5 kg) hammer which drops through a height of 30" (76cm) and pushes a sampling tube of 2"(5.1cm) diameter through a distance of 12" (30cm). 100% efficiency is assumed when the hammer is in free-fall. However, in real tests, the hammer is never in free-fall and the mechanism that is used to lift the weight (such as a rope going around a pulley) and allowed to fall (rope released) loses some energy. The most common practice gives about 60% efficiency. But different practices have different efficiencies. SPT is performed at intervals while drilling for site investigation is in progress. For correlation purposes, the measured SPT value (N) at a given depth is normalised to an effective overburden pressure of 100kPa (denoted N_1) and to an efficiency level of 60% (denoted N_1^{60}), Seed et al (1985). Thus

$N_{60} = N.\ E/60$ (Normalisation with energy efficiency)

$N_1 = N.C_N$ (Normalisation with depth)

$N_1^{60} = N.\ C_N\ E\ /\ 60$ (Normalisation with depth and energy efficiency

where

C_N = Depth normalisation factor = $(100/\ \sigma')^{0.5}$. Another expression for C_N is given by $C_N = 0.77 \log_{10} (2145/\sigma')$

E = Energy efficiency of the SPT procedure (%)

σ'= Overburden pressure at the point of measurement (kPa).

CPT is a continuous process and in this case a device which has a (10cm^2 base area, 60^o) cone at the tip is pushed through the ground at a constant rate (2cm/s) and the resistance is measured continuously. The device has capabilities to measure other parameters, such as pore pressures.
For study of stratigraphy of the soil, the CPT is preferred now a days. Similar to SPT, the CPT values are normalised to an effective overburden pressure of 100 kPa.

SPT and CPT values are empirically related to other soil parameters. Seed et al(1986) gives:

$$V_s\ (m/s) = 85\ N_{60}^{0.17}\ (D_m)^{0.2} \quad (1)$$

V_s = Shear wave velocity in the soil = $(G_{max}/\ \rho)$

D_m = Depth of measurement in metres.

G_{max}= Shear modulus at low strain

ρ = Density (mass)

There are other similar relationships, mainly relating to G_{max} e.g. Imai and Tonouchi (1982), Stroud (1988), Wong and Pun (1997), Baldi et al (1989), Rix and Stokoe (1991), Mayne and Rix (1993).

There are other field and laboratory measuring devices which are not very common yet.

1.2.3 Model Tests:

f) Shake table tests

g) Centrifuge tests

In these tests, models of soil structures are subjected to loading conditions that are expected in the field and the behaviour is analysed. From the response of the model, the behaviour of the prototype is interpreted, based on the theory of models.

1.3 Behaviour of soil under cyclic/dynamic loading

Modelling the soil behaviour requires the understanding of soil properties under cyclic/dynamic loading conditions.

Cyclic loading of dry soils (or saturated soil under drained conditions) does not show any change of strength with cycles. See figure 4. There is no degradation of strength. The behaviour of the soil in the monotonic loading can be used to interpret its behaviour under cyclic loading. Behaviour of saturated soil is, however, dependent on the type of soil. Sand or silt or clay all show different behaviour, particularly in undrained conditions. In sand, behaviour also depends on the relative density while in clay the behaviour depends on whether the soil is normally or over consolidated and also on its plasticity index.

Fig. 4 . Simple shear drained cyclic loading test.
(After Sha'al, 1972)

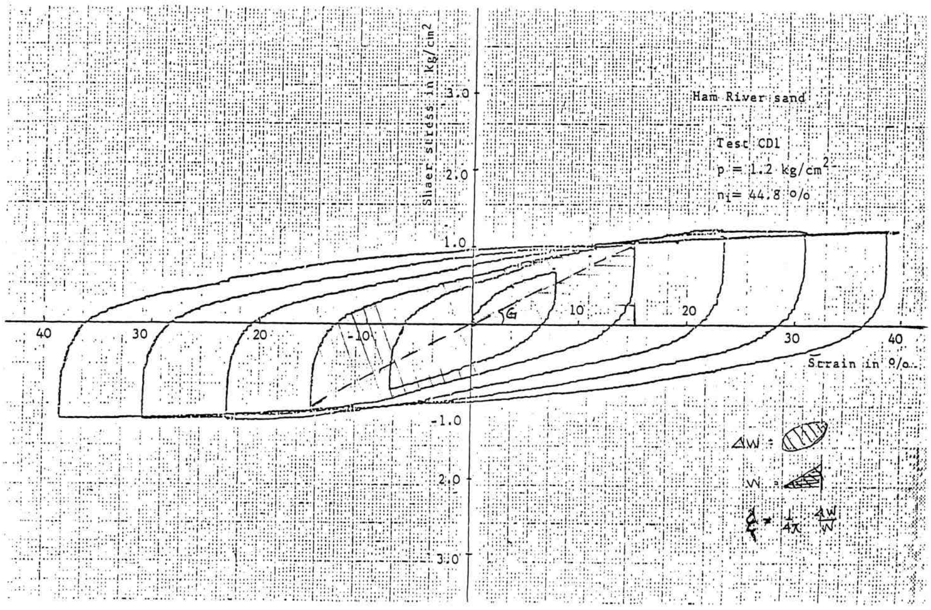


Figure 4: Simple shear drained cyclic loading test (after Sha'al 1972)

1.3.1 Cyclic pore water pressure rise

In order to understand the behaviour of soils under undrained loading, we need to understand the two kinds of soils

(a) Collapsible soil- In this case, pore pressures rise continuously; at first the strength increases reaching a peak and then decreases until a steady state (residual) strength is reached which is lower than the consolidation pressure. This is a strain softening material.

(b) Non-collapsible soil- In this case, pore pressures rise continuously until a state is reached beyond which, pore pressures tend to decrease. The strength increases after failure. This is strain hardening material.

The behaviour of saturated soil under cyclic loading in undrained conditions can be understood from the stress-strain diagrams as shown in the figure 5 and from the stress path diagrams shown in figures 6. This shows that as the soil is first loaded from the initial confined state, pore pressure develops, particularly in loose cohesionless soils and in normally consolidated clays. In dense cohesionless soils or in over consolidated clays, we may have initial negative pore pressures even.

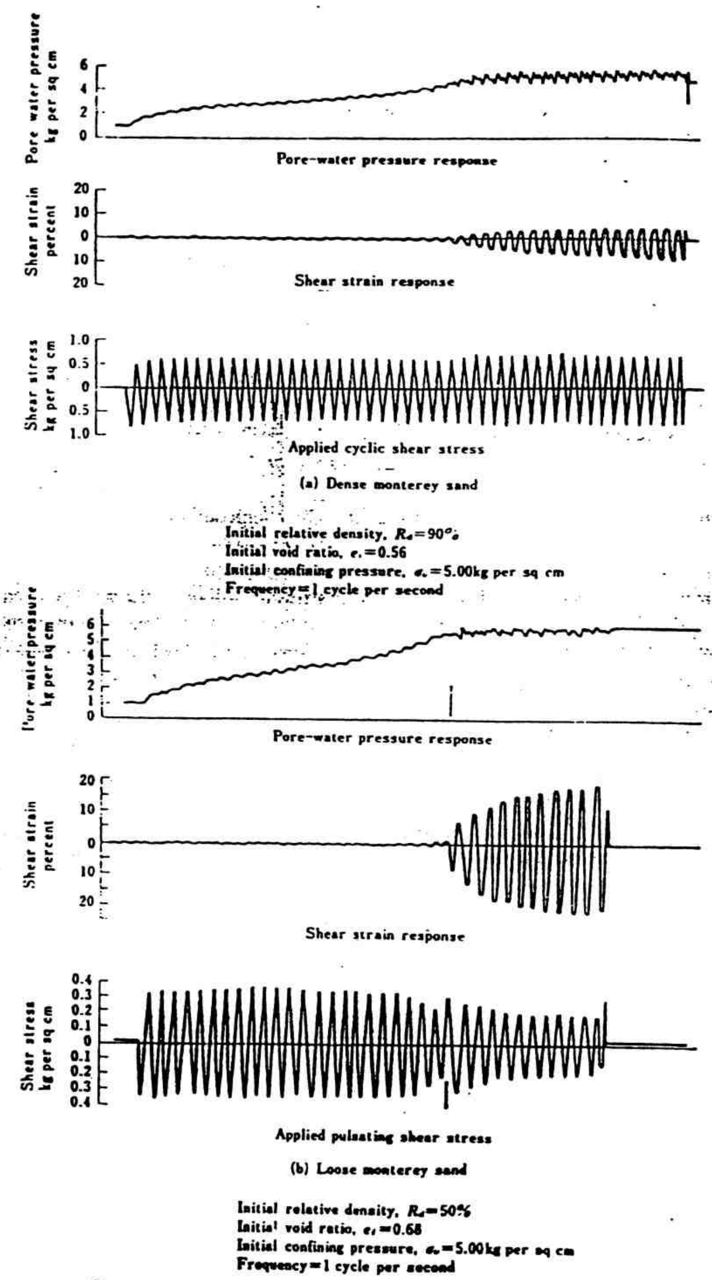


FIG. 5 **Record of a typical pulsating simple shear test of (a) dense sand and (b) loose sand (after Prof. H. B. Seed).**

However, during cycling, there will be positive pore pressures. On unloading, the pore pressure is not reduced. But on reloading, whether on the same or in opposite direction, more pore pressure develops and therefore the ultimate strength and the stiffness of the soil reduces. Therefore, due to the increase of the pore pressures, the effective stress approaches the failure envelope. When it is close to the failure envelope, large strains accumulate. For very loose cohesionless soils, the soil structure may collapse and is unable to withstand any external pressure. The behaviour may be different in stress-controlled and strain controlled tests. While in a strain controlled tests, we will see the steady decrease in strength showing liquefaction, in a stress controlled tests, a large cumulative strain will be witnessed in a very small time. In loose cohesionless soils, the increase of pore pressures may lead to liquefaction (a state of zero effective stress). Monotonic loading after such liquefaction will show nearly zero shear strength. In dense cohesionless soils or in clays or silts, liquefaction (in the sense of zero effective stress) is unlikely to happen or may happen momentarily. However, cyclic loading may lead to large cumulative strain (which is same as failure).

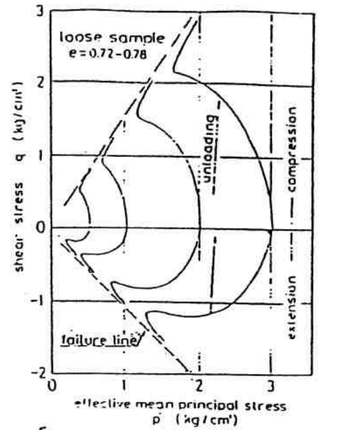


Fig. 6 Typical undrained stress paths for loose samples (Fuji river sand) (After Ishihara *et al*, 1975)

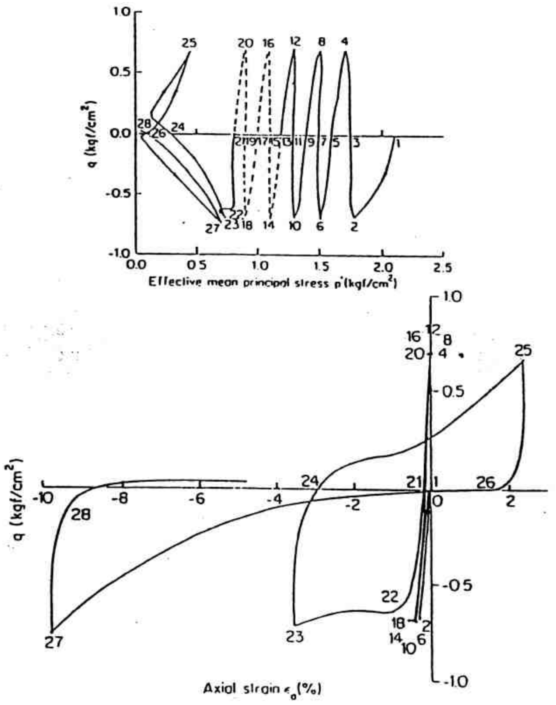


Figure Cyclic triaxial undrained compression-extension test on loose Fuji River sand (after Tastuoka) . Initial $e = 0.737$; confining pressure $\sigma_r = 2.1$ kg/cm²

(After Wood, 1982)

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For dense or medium dense sand consolidated under low confining pressures- even though cycling may produce momentary liquefaction (zero effective stress)- monotonic loading after cycling shows considerable strength. Real liquefaction does not happen but there may be large cumulative strain which is unacceptable.

For very loose sand, the structure of the sand may collapse due to cyclic loading (or may be even due to monotonic loading - the phenomenon of quicksand) and any monotonic loading after such collapse may show nearly zero shear strength. This is proper liquefaction.

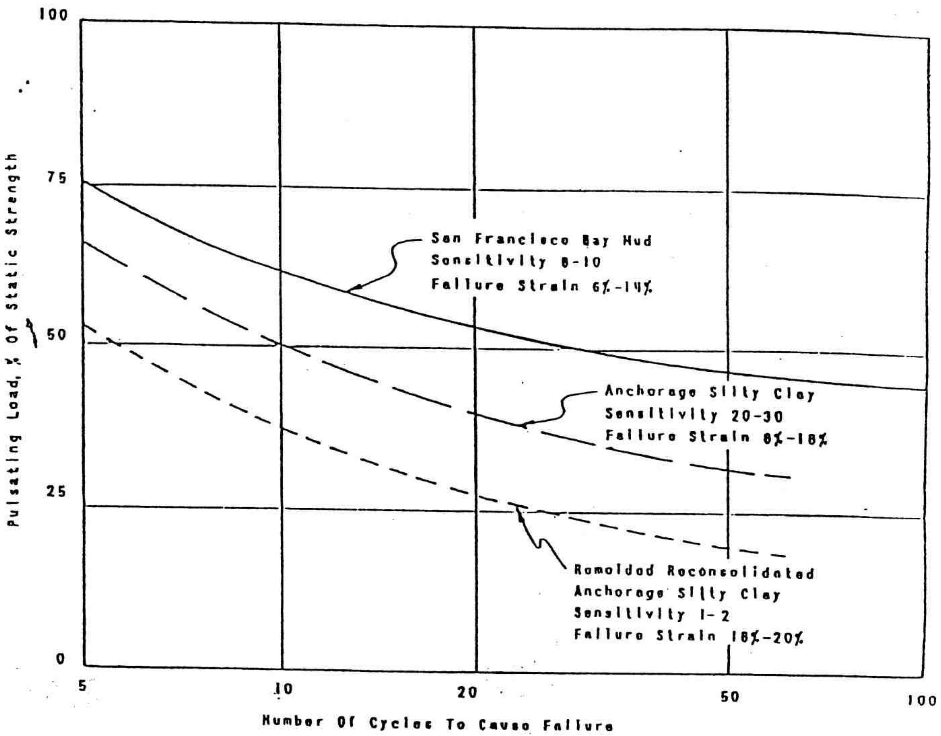

Fig.7 Comparison of results of pulsating load tests.
(After Thiers & Seed, 1969)

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The rise in pore pressures and the cumulative strain depend on the level of the cyclic stress compared to the static strength. See figure 7. For the ratio $\tau/s_u > 0.6$, (τ is the applied shear stress and s_u is the limit shear strength, given by the maximum deviatoric stress) the cumulative strain approaches failure within about 10-20 cycles or less. For smaller ratios, the pore pressures and the cumulative strain approach a limit value. For cycling within a very small strain level, the pore pressures and the strain appear to be elastic.

Also the cyclic strength depends on the initial static stress that a soil sample may be subjected to. While, in a horizontal layered soil deposits, the cycling generally loads the sample in both directions (i.e stress reversal), a soil sample under an embankment may be initially stressed in such a way that the cycling may load the sample only in one direction i.e no stress reversal. Behaviour of a soil sample may be different under these two circumstances.

In case of clay soil, cyclic loading does not produce liquefaction but may produce large cumulative strain. There is a rise of pore water pressure as in the static undrained loading and the cycling produces extra pore pressures at a very slow rate and therefore is of little consequence during earthquakes. For such soils, the static undrained strength becomes important and does not change very much with smaller number of cycles.

In soils, it is noticed that the volumetric strain or the excess pore pressure increases simply by the rotation of the principal stress directions without changing their values. (This effect is not properly quantified yet.). Therefore, it is essential to reproduce the field stress state as faithfully as possible in order to determine the soil strength.

1.3.2 Strain-rate/ Displacement rate effect

In case of sand, the strain-rate has little effect on the strength but in clay, the speed of loading has considerable effect. (This implies viscous damping.)

Even when the soil is at its residual strength statically, during monotonic rapid loading it shows an initial gain of strength leading to a peak and then reduce to a fast residual strength value. The fast residual strength may be bigger or smaller or same as the static value, depending on the type of material, Parathiras (1994), Tika (1989), Lemos (1986). This may explain why slopes which exist at its residual value before earthquake does not fail during earthquake because of the initial gain of strength. However, if the earthquake is strong enough to produce sufficient displacement, it may go past the peak followed by catastrophic failure.

1.4 Liquefaction

In literature, the term liquefaction is quite often used to define either zero effective stress or large strain situation.

Liquefaction of soils leads to catastrophic failure. On level ground, this leads to loss of bearing capacity. On sloping ground, this leads to flow conditions. On very mild slopes, lateral spreading occurs.

There may be some delay for the effect of liquefaction to appear on the surface. This happens when the liquefied deposits are at some depth overlain by a relatively impermeable clay layer. It takes time for the water under high pressure at the depth to flow out through the clay layer thus effecting the pore pressures in the top layer and causing damage. The upper layer will first swell and then consolidate while the liquefied layer will consolidate. The time and the pressure gradient will depend on the relative consolidation and swelling characteristics of the two layers, Ambraseys and Sarma (1969), see figure 13.

A discussion on liquefaction is given in Section 2.

1.5 Pore pressure parameters A & B

Skempton defined the two pore pressure parameters such that

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)] \tag{2}$$

where

Δu = change in pore pressure

$\Delta\sigma_1$, $\Delta\sigma_3$ = changes in major and minor principal stresses.

B depends on the degree of saturation. B equals 1 for fully saturated soils and 0 for dry soils.

A depends on the stress path to failure.

Sarma & Jennings (1980) defined the parameter A_n as a function of the number of cycles. Figures 8a and 8b shows the rise of pore water pressure with cycles.

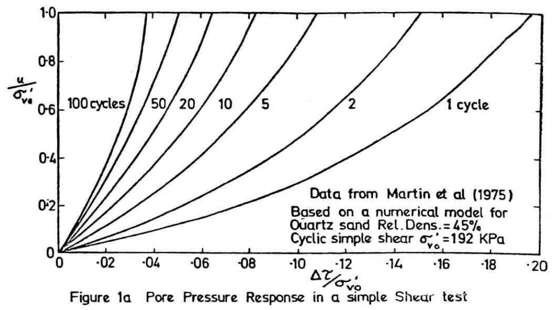


Figure 1a Pore Pressure Response in a simple Shear test

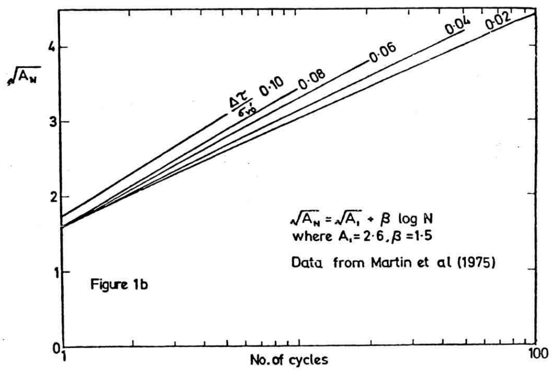


Fig. 8a: Pore Pressure Parameters under cyclic loading

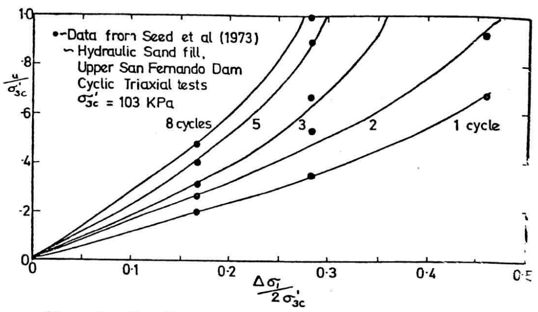
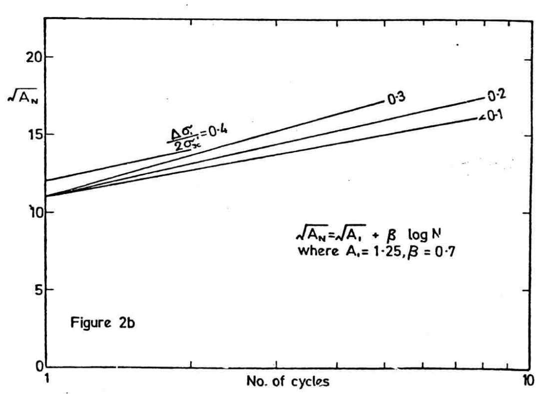


Figure 2a Pore Pressure response in a cyclic triaxial test



1.6 Undrained Strength

Using the parameter A_n with B=1, from the Mohr's Circle of stress at failure in terms of effective stresses and assuming that there is no effect of principal stress rotation on the pore pressure, we can define the undrained strength of the soil c_u (or s_u).

$$c_u = \frac{c'\cos\phi' + p'[K_0 + A_n(1-K_0)]\sin\phi'}{1-(1-2A_n)\sin\phi'} \tag{3}$$

Where p' = Major confining stress
K_o = Coefficient of anisotropic consolidation. (K_o p' = Minor confining stress)
c_u = Undrained peak strength
A_n = Pore pressure parameter which depends on number of cycles and on the soil type and the stress path
c' and ϕ' = Effective strength parameters of the soil

Undrained shear strength can be estimated empirically through the SPT value, e.g. Stroud(1988).

LIQUEFACTION PHENOMENON

2.1 Introduction

What is liquefaction: When a soil loses its shear strength due to pore water pressure, the soil is said to liquefy. In the literature, liquefaction is also associated with large strains and deformations. In this sense, the soil does not lose its shear strength completely but is unable to support a structure on it.

Cause of liquefaction: The cause of liquefaction in the field is the collapse of the structure of loose saturated cohesionless soil due to vibration and cyclic loading. Under static condition, liquefaction is associated with the "quick" condition.

Liquefaction phenomenon manifests in several forms:

Sand Boils: Sand boils are evidence of high pore water pressure at some depth which is generally caused by liquefaction.

Flow failures: Caused by liquefaction of loose cohesionless deposits in or around sloping ground causing massive land slides. These most commonly occur in mine tailings and may also take place without earthquakes.

Lateral spreading: These generally develop on gentle slopes due to liquefaction of subsurface deposits. In this case, ground breaks up causing large fissures and similar phenomenon and the ground generally moves slowly down the slope.

Ground oscillations: When the ground is very gentle, lateral spreading may not occur but the ground may still break up into blocks and these blocks may oscillate on the liquefied layer causing opening and closing of fissures.

Bearing capacity failure of structures: Due to liquefaction, the bearing capacity of the ground below a foundation decreases causing tilting or sinking of structures.

Rise of buried structures: If buried structures are lighter than the liquefied deposits, then these may float upwards in the liquefied soils.

General ground settlements: This is a post earthquake phenomenon which occurs due to densification of deposits after liquefaction.

Failure of quay walls: Due to liquefaction, the pressure on the wall may increase causing failure of retaining walls.

2.2 Assessment of liquefaction potential

The method of assessing the liquefaction potential by field performance was proposed by Seed et al (1983) and later modified by Ambraseys (1989). In the following section, Ambrasey's procedure is followed and in the attached paper, a more detailed version is given, Sarma(1999).

This is an empirical method determined from field evidence of liquefaction in earthquakes.

The liquefaction potential of a site with loose cohesionless deposit depends on

a) The size of the earthquake generally measured by the surface wave magnitude or the moment magnitude preferably the moment one;

b) The distance of the earthquake from the site measured as epicentral distance or fault distance, preferably the fault distance;

c) The SPT value of the cohesionless soil deposit, which is normalised to an effective overburden pressure of 100kPa and a rod efficiency of 60%. This is denoted by N_1^{60}.

d) The position of the water table with respect to the deposit.

Given the size of an earthquake, liquefaction was observed up to a distance, given by the relationship

$$M_w = 4.68 + .0092\ R_f + 0.90 \log R_f \text{ -----------(4)}$$

where R_f is the fault distance in km,[Ambraseys(1989) which is modified to represent the distance in km]. Figure 9. This does not mean that liquefaction cannot occur at further distances but only that no liquefaction has been observed.

Attenuation relationship, Joyner and Boore(1981), gives corresponding values of peak acceleration expected at a distance.

$$\log (a_g) = -\ 1.02 + 0.249\ M_w - \log (r) - 0.00255\ r + 0.26p \text{ -------(5)}$$

where $r^2 = (R_f^2+53.3)$ and p is zero for 50 percentile values and 1 for 84 percentile values of a_g. Figure 10 shows the expected peak ground acceleration values at the limit distance. We can use any other attenuation relationship. This shows that level of acceleration required to produce liquefaction is not a constant but varies with the magnitude and the distance of the earthquake.

A relationship between the cyclic stress ratio Q and the SPT value of liquefied sites (at the boundary between liquefied and non-liquefied sites) was observed which gives, see figure 11:

$$Q = 0.4 \exp(0.06N_1^{60})(N_1^{60})^{0.755} \exp(-0.525M_w) \text{ ------ } (6)$$

where

$$Q = \tau/\sigma_o' = 0.65\,(a_{max})[\gamma h/\sigma_o']r_d \text{ ------------------------}(7)$$

where a_{max} is the peak acceleration at the ground surface (in the absence of liquefaction) given as a fraction of g. The depth is h. The Q in equation 6 represents the cyclic resistance ratio (CRR) while Q in equation 7 represents cyclic stress ratio (CSR).

The above equations are combined to give the set of graphs shown in figure 12, which allows the liquefaction potential to be assessed. These curves are valid for overburden pressures of less than 1.2 kg/cm^2.

N_1^{60} represents normalised SPT value, discussed in section 1. The measured N value at any depth is normalised to an effective overburden pressure of 100 kPa (or approximately 1t/ft^2) which gives N_1 and then normalised to 60% efficiency of the SPT procedure.

The relationship is given by:

$$N_1^{60} = N.C_N E/60$$

where

$$C_N = \sqrt{\frac{100}{\sigma_0^{/}}}$$

E = Efficiency of the SPT procedure

$\sigma_0^{/}$ is the effective overburden pressure at the depth measured in kPa.

Note: 100 kPa represents approximately 10 m depth of submerged soil.

r_d represents the effect of the response of the soil layer on the average shear stress at depth. If the soil is rigid, then the accelerations at any depth will be equal to that at the surface. The value of r_d in that case will be equal to one. Since, soil is not rigid, the induced average stresses at a depth will be slightly reduced. The general practice is to vary r_d linearly from 1 at the surface to 0.9 at 10m depth. Actual variation will be a curve.

The maximum acceleration, a_{max} , generally occurs about once or twice during the earthquake. The average acceleration in the time history is about 0.65 a_{max} which corresponds to about 95% energy of the record.

The reason that large magnitude earthquake may liquefy sites at large distance even though the corresponding acceleration is small is due to the longer duration and therefore to the larger number of cycles. Liquefaction can be achieved by smaller number of cycles with large stress amplitude and by larger number of cycles with smaller stress amplitude which is borne out by laboratory tests as well.

Figure 12 combines all the information given above and allows easy determination of liquefaction hazard for a given seismic source.

2.3 Methods of improving liquefiable sites:

Following techniques may be considered.
a) Remove and replace unsatisfactory material.
b) Densify the loose deposits
c) Improve material by mixing additives
d) Grouting or chemical stabilization
e) Drainage solutions.

2.4 Post-seismic failure due to liquefaction

Liquefaction of a loose sand layer underneath a competent soil layer may delay the effect of liquefaction to be visible on the surface. The consolidation of the liquefied layer with time and the swelling and consolidation of the top layer may increase the pore pressure in the top layer some time after the earthquake and may cause bearing capacity failure. The upward flow of the soil may even cause piping failure. The delay depends on the relative consolidation and swelling properties of the two layers. This characteristic is shown in figure 13, Ambraseys and Sarma(1969).

6. References:

Soil Properties

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SHEAR WAVE VELOCITIES IN FOUNDATION AND BUILDING MATERIALS
(S- Wave velocities in m/sec)

Material	Depth	of	Deposit
	1 - 6 m	7 - 15 m	> 15 m
Loose sand, saturated	60	-	-
Fluvial sand	60	100	125
Clay	60	200	300
Silt	60	-	-
Silty clay	60	240	-
Marsh-land	80	-	-
Reclaimed land, recent	50	100	-
Sandy clay	100	250	-
Gravel , loose	100	300	600
Fine sand, saturated	110	-	-
Medium sand, uniform grading	100	140	-
Tertiary moist clay	130	-	-
Clay mixed with sand	140	-	-
Loam	150	200	-
Dense ssand	160	-	-
Saturated medium sand	160	-	-
Argillaceous sand	170	-	-
Gravel with stones	180	-	-
Clay saturated	190	-	-
Medium sand with fines	190	-	-
Clayey sand with gravel	200	-	-
Medium sand in-situ	220	220	-
Marl	220	-	-
Dry clay with limestones	220	-	-
Compacted clay fill	240	-	-
Dry loess	260	-	-
Puddled clay heavily compacted	-	320	-
Coarse gravel tightly packed	420	-	-
medium gravel	-	330	-
Quartz sandstone	-	-	780
Atlantic muck, ooze	-	-	1000-1500
Hard sandstones (mesozoic)	-	-	1200
Ice, glaciers	-	-	1600-1700

Tuffaceous sandstone	-	-	2000
Concrete (= 0.21, 137 pcf)	-	-	2200
Mesozoic shales	-	-	2350
Granite (intact)	-	-	2700
Limestone (palaeozoic)	-	-	3420
Clay slate (palaeozoic)	-	-	3610