CHARACTERISTIC RESPONSE OF SOIL TO EARTHQUAKE LOADING

by

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ABSTRACT

Liquefaction induced damage together with early studies by Casagrande and Seed into the characteristic response of sand under seismic loading are reviewed. Seed and his co-workers have been addressing the question - what does it take to trigger liquefaction or cyclic mobility and what are the likely strains?, while Casagrande and Castro have been addressing the question of the steady state or residual strength for use in evaluating stability.

The results of more recent monotonic and cyclic loading tests are presented in some detail. The concepts of "collapse surface," "phase transformation" and "steady state" lines are examined for both monotonic and cyclic loading and are seen to be common to both types of loading. The triggering of liquefaction or cyclic mobility is related to the collapse surface or phase transformation line, while the residual or steady state strength is related to the steady state line.

It is seen that Casagrande and Seed were each addressing two separate halves of the problem and the complete solution involves the amalgamation of both their concepts.

INTRODUCTION

Liquefaction of soil has led to very severe damage to structures during past earthquakes. Recorded damage dates back to 2000 B.C. in China. However, there has been damage at more recent events such as Niigata, Japan 1964; Alaska, 1964; San Fernando, California 1971; and China, 1976 that has stirred research and development in the area of soil liquefaction.

The types of damage that have occurred due to liquefaction include:

- Sand boils;
- Flow failure of slopes such as occurred at the lower San Fernando dam in California in 1971;
- Settling and tipping of buildings as occurred at Niigata (Fig. 1);
- Retaining wall collapse;
- Lateral spreading of slopes;
- Settlement and flooding;
- Rise of buried structures such as tanks (Fig. 2).

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The damage in these cases was caused by a severe loss in stiffness and/or strength which is commonly termed liquefaction. It can occur in loose to compact saturated sands, silts and gravels and is due to a rise in porewater pressure caused by the earthquake induced shaking.

Our understanding of liquefaction comes from:

- Laboratory studies on both soil samples and soil models;
- Field observations at sites that have been subjected to earthquakes; and
- Theoretical studies and analyses.

The shearing strength, s, of granular material (Fig. 3) can be expressed by the equation:

$$s = (\sigma - u)\tan\phi' = \sigma'\tan\phi' \qquad (1)$$

in which:

σ = the total normal stress
u = the porewater pressure
σ' = the effective normal stress, and
ϕ' = the effective angle of internal friction

The total normal stress, σ, is generally known. The unknown in the earthquake problem is the pore pressure, u. During earthquake shaking, u, can rise to equal, σ, in which case the resistance drops to zero and the sand may undergo large deformations, and this is commonly referred to as liquefaction. Herein the early studies on liquefaction by Casagrande and Seed will first be reviewed, followed by recent advances that have occurred in the past 10 years.

EARLY STUDIES BY CASAGRANDE AND SEED

For the normal or static loading case, the sand can freely drain and the porewater pressure, u, is known from the groundwater conditions. This drained strength, s_d, can be obtained from Eq. 1. For the earthquake condition it can be argued that there is no time for drainage to occur and so the undrained strength, s_u, is appropriate. This was the approach taken by Casagrande (1936). His concept is shown in Fig. 4a where it may be seen that a sand can have a void ratio, e, between the limit states e_{max} and e_{min} under any stress, σ_o'. However, at failure, Casagrande found a unique relationship between void ratio and effective stress as shown. This failure state line he called the critical state line, and it controls the strength of the sand as shown in Fig. 4b.

If the sample is loose and has a void ratio, e_L, that lies above the critical state line (CSL), then, if sheared drained, it will reduce in volume along the path C-D to end at e_{crit}. If it is sheared undrained, it will follow the path C-U, generating excess porewater pressure to end at a lower σ' and hence a lower strength as shown in Fig. 4b.

If the sample is dense and has a void ratio, e_d, that lies below the CSL, then, if sheared drained, it will expand at constant effective stress,

σ_o', to reach the same e_{crit} as the loose sample. If it is sheared undrained, it will follow the constant void ratio, e_d, to reach the CSL at a much higher σ' and hence a much higher strength, as shown in Fig. 4b.

Casagrande considered that, provided the undrained strength, s_u, was greater than the drained strength, s_d, the sand would not lose strength due to earthquake shaking and would therefore be stable. For this to be true, the field void ratio must lie below the critical state line.

Casagrande was concerned about the uniqueness of the CSL for drained and undrained conditions and also on the importance of strain rate effects. Casagrande (1975) considered that high strain rates such as occurred under load controlled conditions could create a flow type structure leading to very low strengths.

Seed and his colleagues at Berkeley spurred on by events at Niigata and Alaska in 1964 presented a different approach to liquefaction in numerous papers in the 60's and early 70's. Their approach was basically a "dynamic stress path" one in which the field stress conditions both static and dynamic are applied to representative laboratory samples and the response observed. From the observed laboratory response, the field behaviour is inferred.

The steps in their analysis are as follows:

1. Determine the in-situ static stress conditions.
2. Determine the dynamic stresses imposed by the design earthquake.
3. Apply both static and dynamic stresses to undisturbed samples in the laboratory and observe the response in terms of pore pressures and strains.
4. From the observed response infer the field performance.

The field and laboratory conditions are illustrated in Fig. 5.

The key factor in such a procedure is the characteristic behaviour of the soil under cyclic loading, and this is shown in Fig. 5b. Under drained conditions the samples undergo a decrease in volume with number of cycles, N_{cy}, as shown in Fig. 5b. More volume change occurs for loose than dense samples but even a dense sample contracts further under cyclic loading. Under saturated undrained conditions a decrease in volume cannot occur because of the very low compressibility of both the water and the solids and, instead a rise in porewater pressure occurs as shown in Fig. 5c. When the pore pressure rises to reach the initial effective stress, the cyclic strains become very large as shown in Fig. 6 and this condition is commonly referred to as initial liquefaction.

The number of stress cycles to cause initial liquefaction depends upon the density of the soil, and the cyclic stress ratio level as shown in Fig. 7a. For a given number of cycles, such as 15, the cyclic stress ratio to cause or trigger liquefaction can be presented in terms of relative density as shown in Fig. 7b. Because strains are small prior to initial liquefaction and large after, Seed contends that triggering of liquefaction is undesirable particularly for loose sand, and therefore, there should be a factor of safety against such triggering.

Seed looked at liquefaction from a pore pressure and deformation point of view. What level of shaking does it take to trigger high pore pressures and significant deformations? The required density to prevent such triggering depends upon the intensity of shaking, and $D_r > 75\%$ may be required to prevent initial liquefaction for high intensity shaking.

Casagrande looked at liquefaction from a strength point of view and was concerned whether the undrained strength was adequate for stablity? In most cases, $D_r > 50\%$ would satisfy this criterion. The magnitude of the shaking does not enter into this approach, nor is the level of displacement addressed.

Much research and investigation has been carried out in the past 10 years, by numerous investigators including: Ishihara et al (1975), Finn (1981), Castro et al (1982), Seed et al (1984), Seed (1986) and Vaid and his students at the University of British Columbia including Chern (1984). Many of these ideas are presented in US-NRC (1985). There is now considerable agreement about the undrained response of sand to static (monotonic) and earthquake (cyclic) loading and this will now be presented.

UNDRAINED RESPONSE-MONOTONIC LOADING

Based upon laboratory testing, saturated sand can have basically 3 different stress-strain responses to monotonic loading depending on its density or void ratio as shown in Fig. 8.

- Type 1 - A liquefaction or strain softening response that occurs in very loose sands.
- Type 2 - A partial liquefaction or strain softening followed by a hardening response.
- Type 3 - A non-liquefaction or hardening response.

In general, the type of response depends upon both the relative density and the effective confining stress, but for rounded sand particles it depends mainly on the relative density. The standard penetration test (SPT) can be correlated with relative density and the expected response type in terms of the normalized $(N_1)_{60}$ is shown in Table I.

TABLE I

SPT $(N_1)_{60}$	Characteristic Response Type
0-4	1
4-10	2
> 10	3

The characteristic behaviour is perhaps best seen in stress-void ratio space as shown in Fig. 9. The peaks for strain softening response appear to lie on a straight line which has been called a collapse surface (Sladen et al, 1985), because once this point is reached in a load controlled test, collapse will follow. The test results of Chern (1984) would indicate that this line passes through the origin, while Sladen et al indicate that it passes through the steady state strength point.

The points at which the stress path abruptly turns from a contractive to a dilative response (peak pore pressure point) on Fig. 8 lie on a straight line through the origin and this has been called the Phase Transformation Line (PT) by Ishihara (1975) and corresponds with the constant volume friction angle or ϕ'_{cv} line. Basically sand is always contractive below the PT line and dilative above it.

In void ratio-stress space, the undrained path is a line of constant void ratio as shown in Fig. 9b and the stress point will move first to the left and then turn at the PT point and move to the right to end at the critical or steady state line (SSL).

The important concepts from monotonic loading are as follows:

1. There is a unique relationship between void ratio and effective stress at failure and hence a unique undrained steady state or residual strength for each void ratio as shown in Fig. 10.

2. For loose sands under undrained conditions a collapse surface exists beneath the ϕ'_{cv} line as shown in Fig. 11, b or c. For stress states beneath the collapse surface, the sand will be stable. For stress states between the ϕ'_{cv} and the collapse surface the sand will be unstable and susceptible to spontaneous liquefaction.

UNDRAINED RESPONSE - CYCLIC LOADING

The undrained response to cyclic loading is closely related to the monotonic response and depends on whether the sand is loose (strain softening) or dense (strain hardening).

LOOSE OR STRAIN SOFTENING

The stress-strain response for a loose or strain softening sample subjected to cyclic loading is shown in Fig. 12a. The monotonic response for the sample is shown as the dashed line. It may be seen that strains accumulate with the number of stress cycles, and when the cyclic stress-strain curve intersects the monotonic strain softening curve, a flow or liquefaction failure will occur because the resistance of the soil is now less than the static driving stress. This situation occurs when the driving stress τ_{st} is greater than the steady state strength, s_{us}. If $\tau_{st} < s_{us}$, then although strains may accumulate with number of cycles, a flow failure will not occur. This behaviour is sometimes referred to as cyclic mobility rather than liquefaction. Liquefaction occurs when the stress state hits the collapse line, while cyclic mobility results when the stress point hits the PT line.

The comparison between monotonic and cyclic behaviour may also be examined in stress space as shown in Fig. 12b where it is found that once the stress point hits the collapse surface from monotonic loading, a flow failure occurs. Thus the collapse surface is a common feature of both monotonic and cyclic loading response. It is apparent for the loose sand conditions depicted here, that the higher the τ_{st} the lower is the τ_{dy} required to trigger liquefaction. In fact, if the soil should exist above the collapse surface in the field, a very small disturbance could cause it to fail in undrained shear, and such failures have occurred and are referred to as spontaneous liquefaction.

DENSE OR STRAIN HARDENING

The stress-strain response of a dense sample subjected to cyclic loading is shown in Fig. 13a. The monotonic response is shown as the dashed line. It may be seen that although the cyclic loading may induce considerable strain accumulation, there is no loss in strength, so that a flow or true liquefaction failure is not a possibility. This is so because the undrained strength of the soil is much greater than the static driving stress, τ_{st}, and the collapse surface is well above the stress point and will not be reached.

The stress path followed is shown in Fig. 13b where it may be seen that with cyclic loading the stress point works its way back to the ϕ'_{cv} or PT line and once it reaches it, essentially moves up and down the line. It should be noted that the effective stress does not drop to zero for this case. It will only drop to zero if the combined $\tau_{st} - \tau_{dy} \leqslant 0$.

The accumulation of strain with number of cycles increases as the stress point approaches the PT line as depicted in Fig. 13c and once the PT line is reached, strain accumulates with each cycle as shown.

If the sample is tested monotonically after cyclic loading it essentially moves up the PT line until it reaches the steady state condition and has a strength similar to the uncycled sample. This is depicted in Figs. 13a and b.

For strain hardening material it turns out that the presence of a static bias may help to reduce the strains (Vaid and Chern, 1983). This is so for two reasons:

1. The stress point may move back more rapidly with cyclic loading when stress reversals occur with low or zero static bias as shown in Fig. 14.

2. In the absence of a static bias the stress point cycles through the origin or zero effective stress point as shown in Fig. 14, and very large strains can occur at these low effective stresses.

SUMMARY OF FINDINGS FROM TESTS ON LABORATORY SAMPLES

The studies on laboratory samples indicate the following:

1. The steady state line and the undrained strength are the same for both monotonic and cyclic loading and are independent of the rate of loading, whether it be strain controlled or load controlled.

2. The state of the sand (τ, σ', e) determines its response to both monotonic and cyclic loading. Referring to Fig. 15 the sand may lie in Zones A, B or C depending on the stress and void ratio state.

If the stress state lies within:

Zone A - The system is stable and a flow slide will not occur. However, cyclic mobility and large strains can occur if the stress point cycles up and down the PT line. The magnitude of the strains depends upon the level and duration of cyclic loading if the stress point hits the collapse surface.

Zone B - The system is potentially unstable and a flow slide can be triggered by cyclic loading.

Zone C - The system is unstable and spontaneous liquefaction may occur.

Basically one would always like to be in Zone A. If in the field the stress state is not in Zone A, then the point P on the diagram and the collapse surface can be moved up by densification so that the stress state now lies within the new Zone A.

FIELD APPLICATIONS

There are three questions to address when attempting to evaluate the seismic response of a structure comprised of, or underlain by, saturated granular soils:

1) What level of cyclic loading will trigger initial liquefaction or significant strain, and if triggered,
2) What strains are likely to occur, and
3) What is the residual or steady state strength?

In principle one could obtain undisturbed samples and test to determine the PT and collapse lines shown in Fig. 15. Provided one lay within Zone A, liquefaction should not be triggered and so a flow failure should not occur. The collapse line or the steady state strength could be used to determine a factor of safety against such a flow failure as suggested by Castro and his co-workers. In addition, the likely earthquake induced deformations could be estimated from:

- A Newmark (1965) displacement type analysis in which the steady state strength is used to determine a yield acceleration;
- A dynamic stress path approach in which samples are subjected to the expected static and dynamic stresses, and the observed strains used to estimate displacements;
- A rigorous dynamic analysis incorporating a comprehensive stress-strain law.

Methods of computing such response will be discussed by Dr. Finn in the next session of his symposium.

In practice, it is very difficult to obtain an undisturbed sample of granular material and determine reliable response data. Loose sands will densify upon sampling and very large corrections in their laboratory undrained strength may be required to represent field conditions. Castro et al (1985), in examining the 1971 slide of the lower San Fernando dam, have suggested that the needed correction is a factor of 20. Predicted response based on such disturbed samples could not be expected to yield reliable results.

Because of the problems associated with sample disturbance, Seed et al (1984) and Seed (1986) have proposed that the response of soil to seismic loading be based largely on field testing and experience during past earthquakes rather than on laboratory tests. He has correlated field experience with the standard penetration value, N, corrected for both confining stress and energy level and termed $(N_1)_{60}$ and produced charts which address the 3 questions of concern posed at the beginning of this section:

1) The stress ratios required to trigger liquefaction as a function of $(N_1)_{60}$ and fines content (% finer than the #200 sieve size) are shown in Fig. 16. If the stress ratio caused by the design earthquake lies above the appropriate line, liquefaction is predicted, while if it lies below the line, no liquefaction is predicted to occur. The lines shown can therefore be considered to represent the liquefaction resistance ratio of the soil. It may be seen that for the same N_1 value, liquefaction resistance increases with increasing fines content. Figure 16 is appropriate for earthquakes of magnitude 7.5 on the Richter scale. For other magnitude earthquakes the stress ratio should be corrected as shown in Table II. This correction is necessary because both field and laboratory observations indicate that liquefaction depends not only on the stress ratio but also on the number of cycles of strong motion, which in turn depends on the magnitude of the earthquake.

TABLE II. Correction Factors for Influence of Earthquake Magnitude on Liquefaction Resistance

Earthquake Magnitude, M	Number of Representative Cycles	$\dfrac{\frac{\tau_{eq}}{\sigma'_o} \text{ for } M=M}{\frac{\tau_{eq}}{\sigma'_o} \text{ for } M=7.5}$
8 1/2	26	0.89
7 1/2	15	1.0
6 3/4	10	1.13
6	5-6	1.32
5 1/4	2-3	1.5

2) If liquefaction is triggered, the resulting strains and possible damage levels are shown in Fig. 17. This figure is based upon both field experience and laboratory tests. It indicates that clean sands having $(N_1)_{60}$ values less than 10, can be expected to behave very poorly if liquefaction is triggered. For the same N_1 values, experience shows that silty sands behave better than clean sands. Seed (1986) suggests

that Fig. 17 is also appropriate for silty sands provided an effective N_1 value is used as follows:

$$(N_1)_{effective} = (N_1)_{measured} + \Delta N_1 \qquad (2)$$

where ΔN_1 depends on the fines content as shown in Table III.

TABLE III. N_1 Correction for Silty Sands

Fines Content, %	ΔN_1
$\leqslant$ 5	0
$\approx$ 15	3
$\approx$ 35	5
$\approx$ 50	7

3) Although sands may initially liquefy under cyclic loading, upon shearing in one direction they may regain much of their strength, and this strength is referred to as the residual or steady state strength. Based upon field experience during past earthquakes, Seed (1986) has proposed a tentative relationship between residual strength and $(N_1)_{60}$ as shown in Fig. 18. For silty sand, the effective $(N_1)_{60}$ based upon Eq. 2 and Table II should be used.

Considerable controversy exists regarding the strength after liquefaction has been triggered. Castro (1976) claims that the undrained steady state strength is the same whether the soil has been subjected to static (monotonic) or dynamic (cyclic) loading. Comprehensive testing carried out at the University of British Columbia and reported by Chern (1984) and discussed earlier indicates that Castro's claim is correct. The undrained steady state strength of Ottawa sand as a function of $(N_1)_{60}$, based upon Chern's laboratory test data is also shown on Fig. 18, where it may be seen that it is much higher than the residual strength proposed by Seed.

Two reasons for this difference could be:

a) Seed's proposed values may be too low since his values are based upon back analysis of observed field behaviour. Because the strains required to reach the residual strength are very large, the field movements may not have been large enough to produce residual strain levels and so the strengths mobilized in his analysis could have been well below the residual value.

b) The undrained steady state tests do not model the field situation. It has been hypothesized that because of variations in the field conditions during or after the shaking, migration of water due to pore pressure gradients could cause some zones to densify while others would loosen, and would have strengths less than the undrained values. Experience at Niigata suggests that much of the foundation movements occurred towards the end of the shaking period after loosening had occurred due to a redistribution of porewater.

In light of this difference of opinion on residual strength, it seems prudent to base the post-liquefaction response on the more conservative findings proposed by Seed.

In dealing with large earth structures such as dams and embankments where high normal stresses and a significant static shear stress bias may exist, Seed (1983) has suggested making a correction to the triggering stress ratio shown in Fig. 16 to account for:

1) a possible reduction in the triggering stress ratio for confining stresses in excess of 1 T/ft^2, and

2) an increase in the triggering stress ratio with increasing levels of static bias, τ_{st}.

This second point is rather controversial since Castro has suggested that the presence of τ_{st} will reduce the level of cyclic stress required to trigger liquefaction. Castro has based his conclusion on the results of tests on loose sands in which τ_{st} causes the stress point to move closer to the collapse surface and for these soils increasing τ_{st} will reduce the triggering stress ratio. Seed has based his correction factor on the results of cyclic load tests on compact to dense sands that undergo cyclic mobility rather than true liquefaction. For these sands the presence of τ_{st} can increase the level at which significant accumulation of strain or cyclic mobility commences. So both Castro and Seed are correct for the sand densities they are discussing. It should be noted that Seed's triggering stress ratio refers to a cyclic mobility condition rather than a "true liquefaction" or flow condition.

SAMPLE DISTURBANCE

The question of sample disturbance can be addressed by:

1) Developing methods to obtain truly undisturbed samples, for example by freezing, or

2) Inferring the in-situ void ratio or some characteristic parameter such as dilation angle or <u>state parameter</u> from an in-situ test such as a nuclear density test, a pressuremeter test, or a cone test. Then allow for disturbance by testing the sample at the appropriate reference state.

A state parameter approach for sand has been proposed by Been and Jefferies (1985). Basically the state parameter, ψ, is the vertical distance in void ratio space above the steady state line as shown in Fig. 19a. Tests indicate that samples with the same state parameter have the same normalized characteristic behaviour. Consequently, when disturbance of a loose sample causes a reduction in void ratio as shown in Fig. 19b, the sample if tested at the in-situ stress state would have too low a state parameter. To obtain the correct response the stress on the sample must be increased until the laboratory sample has the same state parameter as the in-situ sand. If the sand in the field has state parameter value, ψ_f, then sample disturbance will reduce this value, and if it is tested at the field stress level, it will have too low a state parameter value and consequently an incorrect response will be measured. To obtain the correct response, the stress on the sample must be increased until the laboratory parameter $\psi_1=\psi_f$ as shown in Fig. 19b.

It is reasonably easy to determine the state parameter in the laboratory, the problem is to find the field state. The nuclear densometer test is a possibility, but the work of Been et al (1986) suggests that ψ_f can be estimated from field cone tests.

By testing the sand in the laboratory samples at the same state as it exists in the field it may be possible to correctly allow for sample disturbance and obtain meaningful data that can be used in analyses.

SUMMARY

Laboratory testing indicates that monotonic and cyclic loading response of saturated sand are all part of the same picture. The behavior depends on the stress state as shown in Fig. 15. If the stress state lies in Zone A, then, although cyclic mobility and significant deformation may result, depending on the level of seismic activity, a flow failure will not occur.

If the stress state lies in Zone B, liquefaction can be triggered and a collapse or flow slide can result.

If the stress state lies in Zone C, spontaneous liquefaction and a flow slide are to be expected.

Prudent design requires that the stress state should lie within zone A. If the field condition is such that this is not so, then densification or drainage would be required to ensure that failure does not occur.

It would be desirable to determine the response of sand within the framework described above, and such an approach is being used by some practitioners. However, this approach required laboratory testing, and sample disturbance is a problem. For this reason, Seed (1986) has proposed that the seismic behaviour of saturated sand be evaluated from the in-situ standard penetration value $(N_1)_{60}$ rather than from laboratory tests. He has presented design charts for estimating:

- the resistance to triggering liquefaction or cyclic mobility, and if triggered,
- the likely strains, and
- the residual strength.

The residual strengths proposed by Seed are much lower than the undrained strength values from laboratory tests, except for very loose sands. It is not clear that the undrained condition commonly assumed in analysis and testing is necessarily the most severe case. Redistribution of porewater during and after shaking may cause some zones to densify while others loosen. Such redistribution may account for the fact that Seed's proposed residual strengths, which are based upon field experience, are considerably lower than would be expected from an undrained assumption.

Because of the uncertainty associated with redistribution of porewater after triggering initial liquefaction, it seems prudent to design against such triggering on critical structures. Field observations, model testing and sophisticated dynamic analysis will hopefully clarify this uncertainty in the near future.

The possibility of correcting for sample disturbance by testing samples in the laboratory at the same state paramater value as the sand in the field looks promising. The "structure" of the sand may also need to be considered in this approach.

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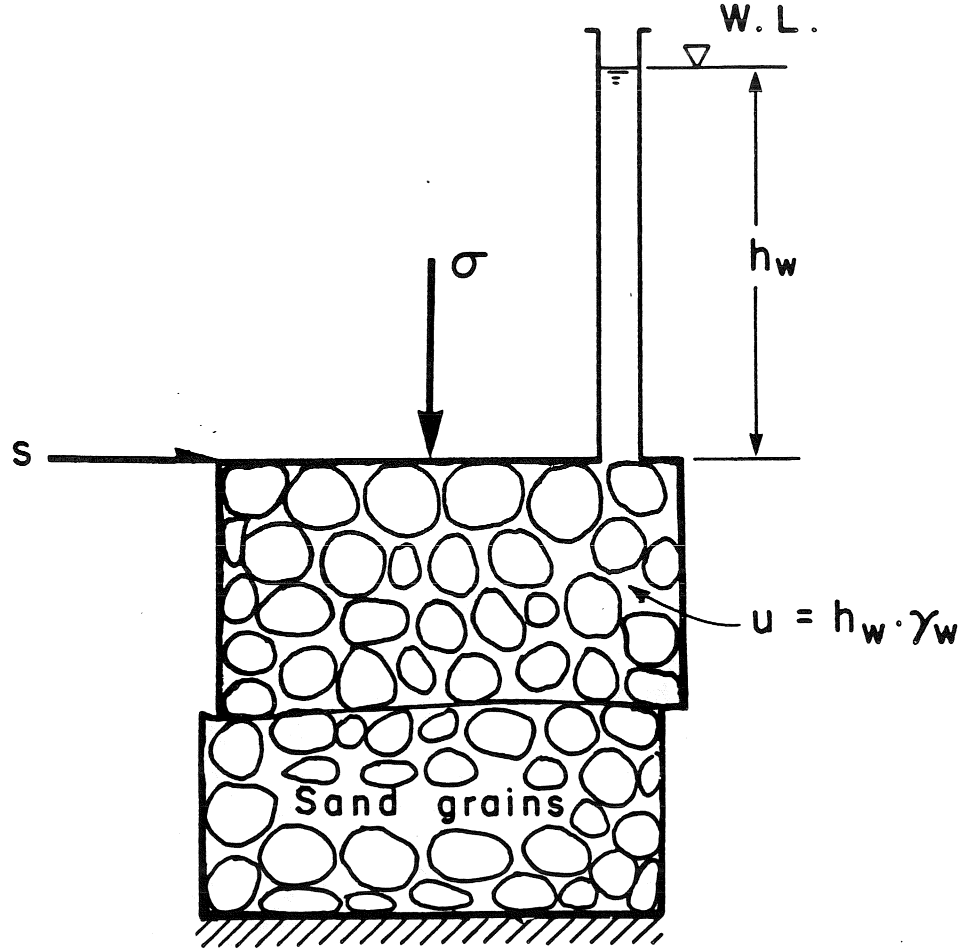
FIG. 1 SETTLEMENT AND TILTING OF BUILDINGS DUE TO LIQUEFACTION OF THE FOUNDATION SOILS AT NIIGATA, JAPAN, 1964.

FIG.2 SEPTIC TANK FLOATED UP ABOVE THE GROUND, NIIGATA, 1964.



$$s = (\sigma - u)\,\mathrm{Tan}\,\phi' = \sigma'\,\mathrm{Tan}\,\phi'$$

FIG.3: SHEARING RESISTANCE OF SAND.

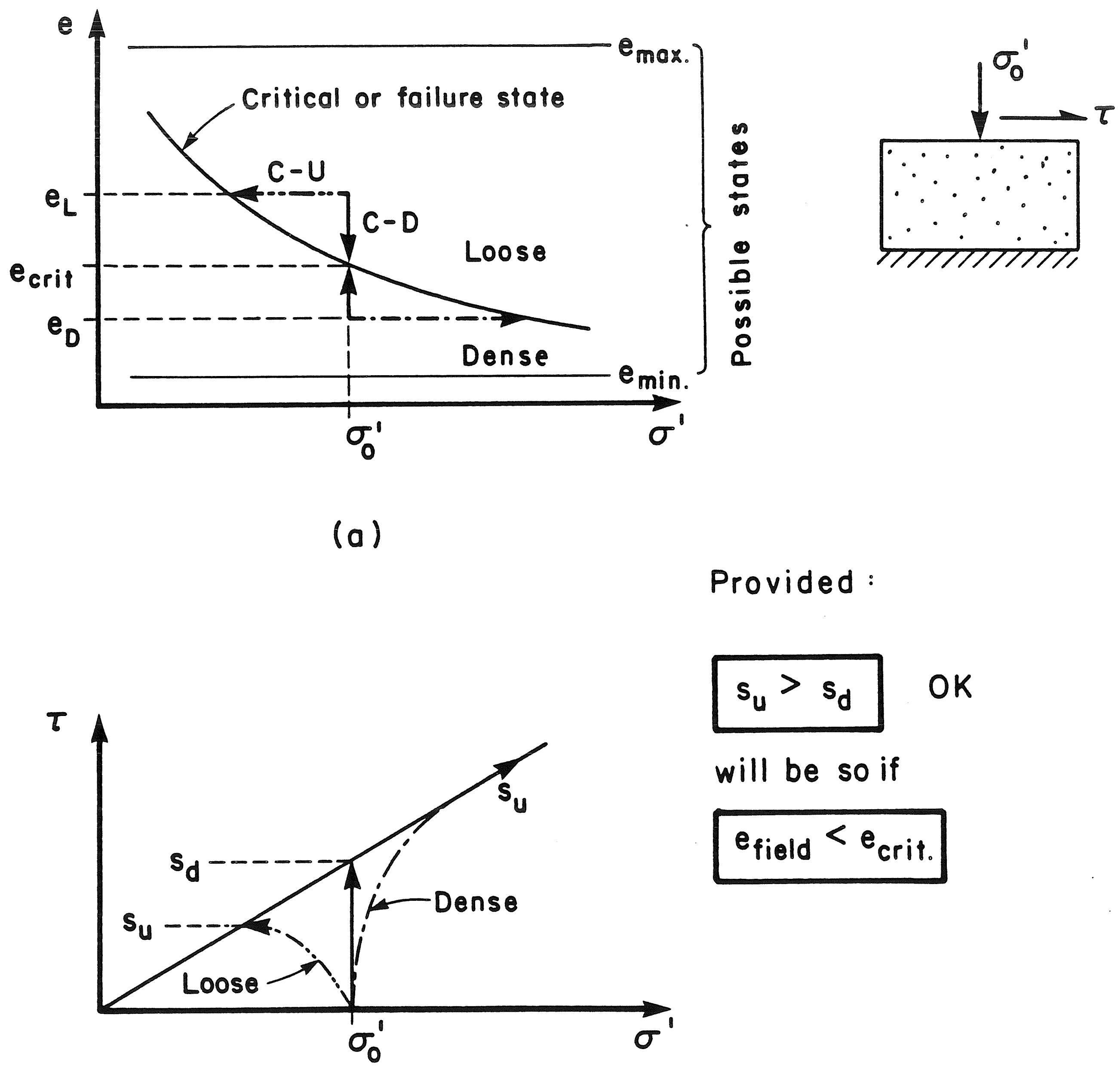


FIG.4: CASAGRANDE CONCEPT OF SAND BEHAVIOR FOR EARTHQUAKE LOADING.

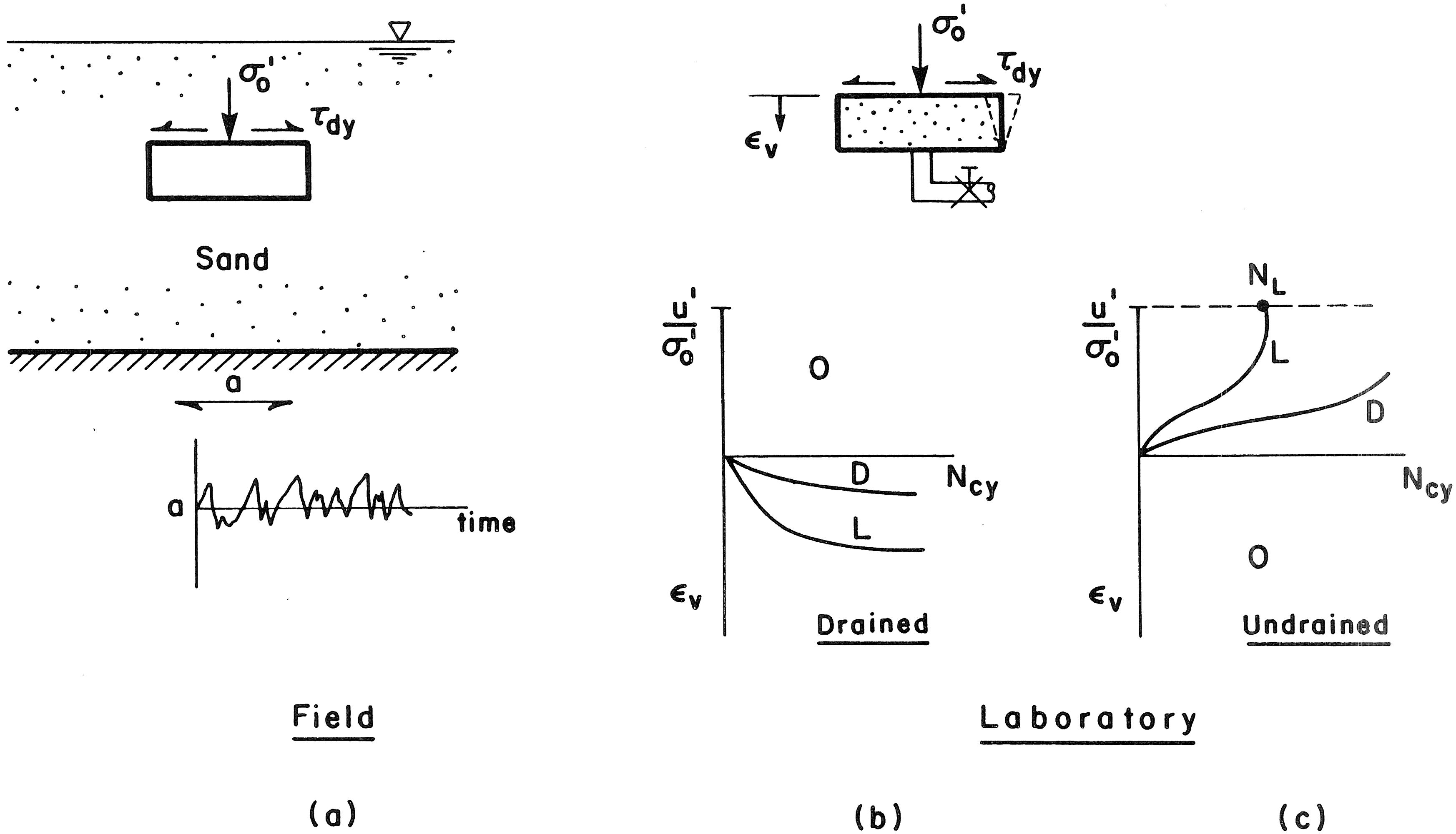


FIG.5: SEED DYNAMIC STRESS PATH APPROACH.

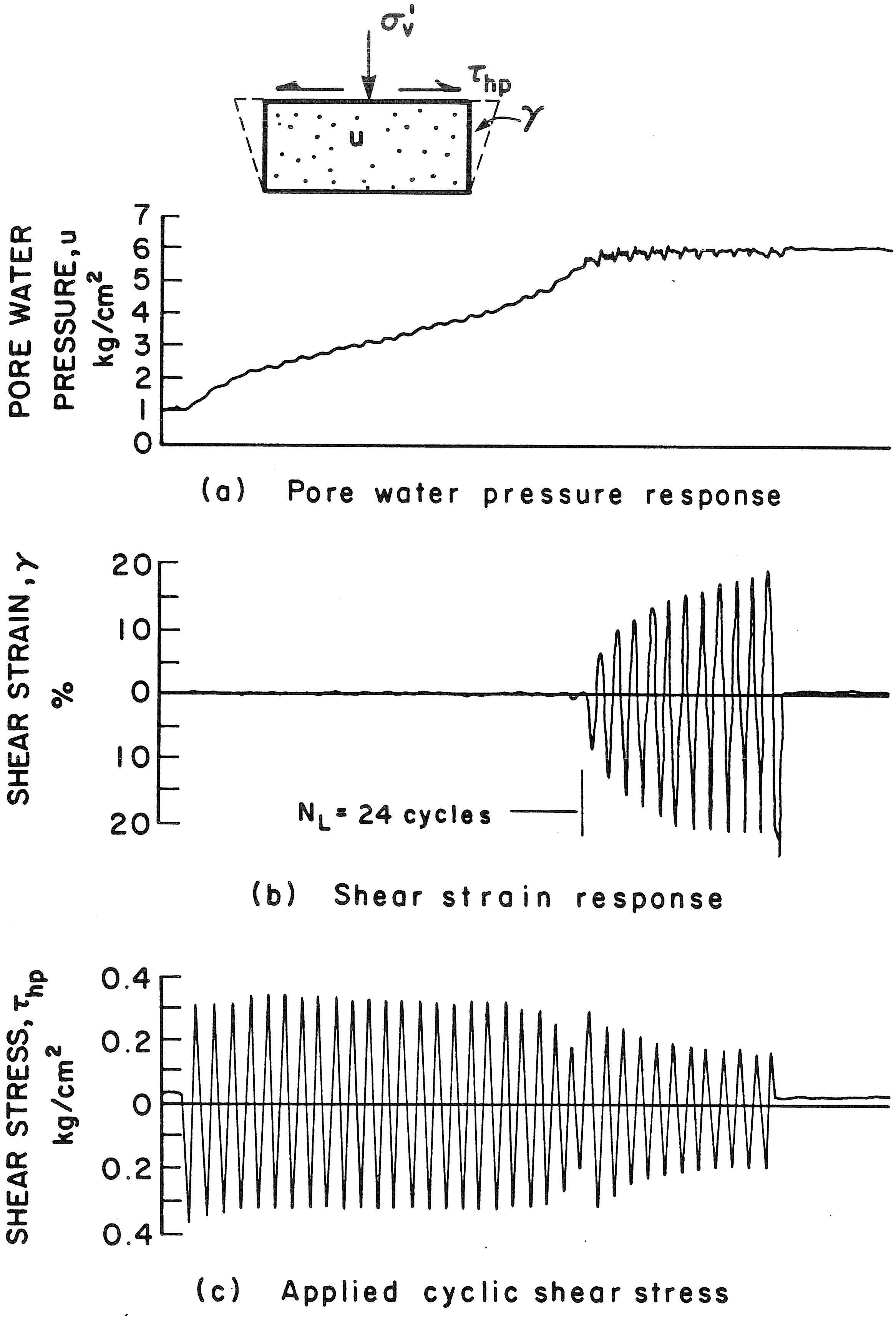


FIG.6: CHARACTERISTIC RESPONSE OF SATURATED SAND TO CYCLIC LOADING.

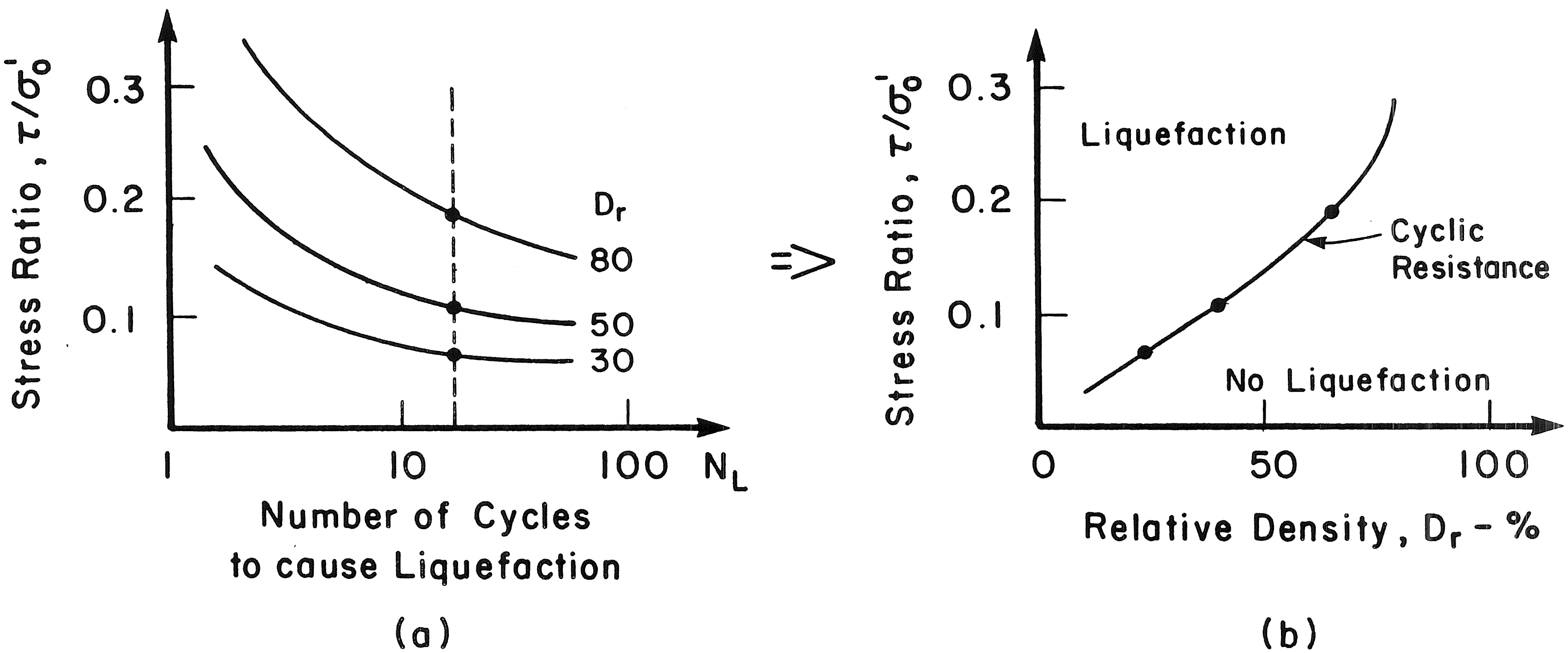


FIG.7: LIQUEFACTION RESISTANCE OF SATURATED SAND.

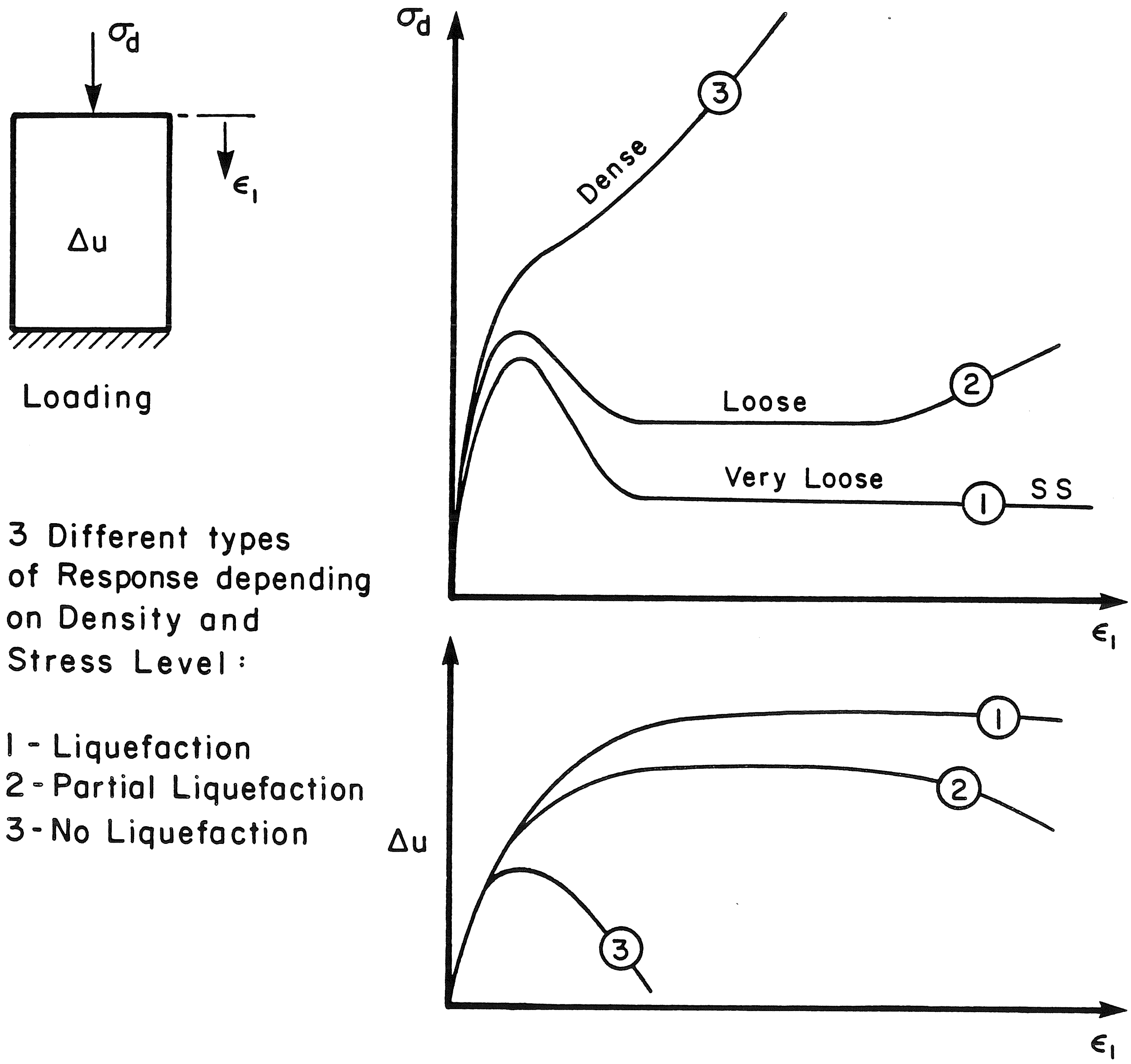


FIG.8: CHARACTERISTIC RESPONSE OF SATURATED SAND UNDER UNDRAINED MONOTONIC LOADING.
(Chern, 1984)

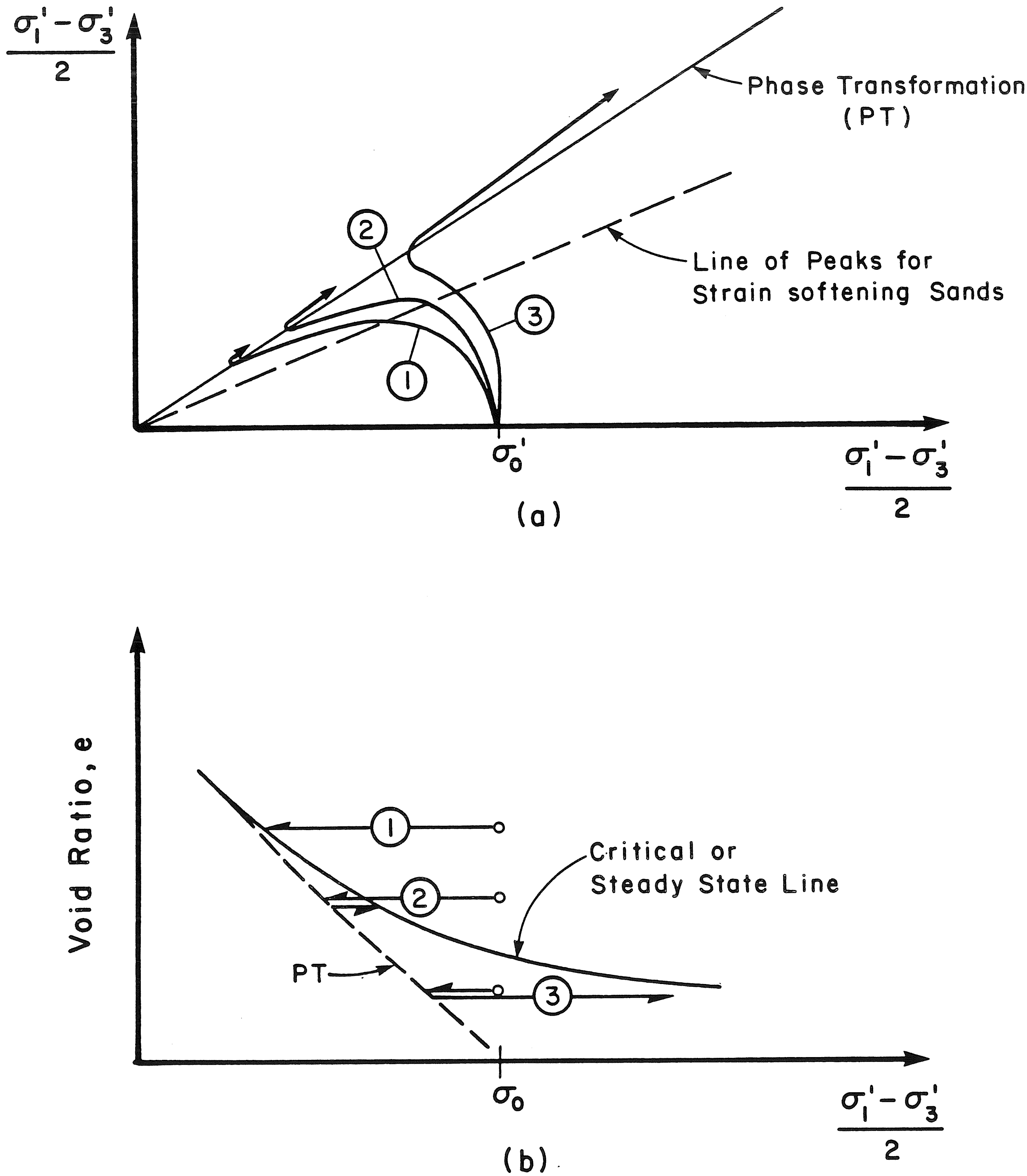


FIG.9: CHARACTERISTIC BEHAVIOUR OF SATURATED SAND UNDER UNDRAINED MONOTONIC LOADING.
(Chern, 1984)

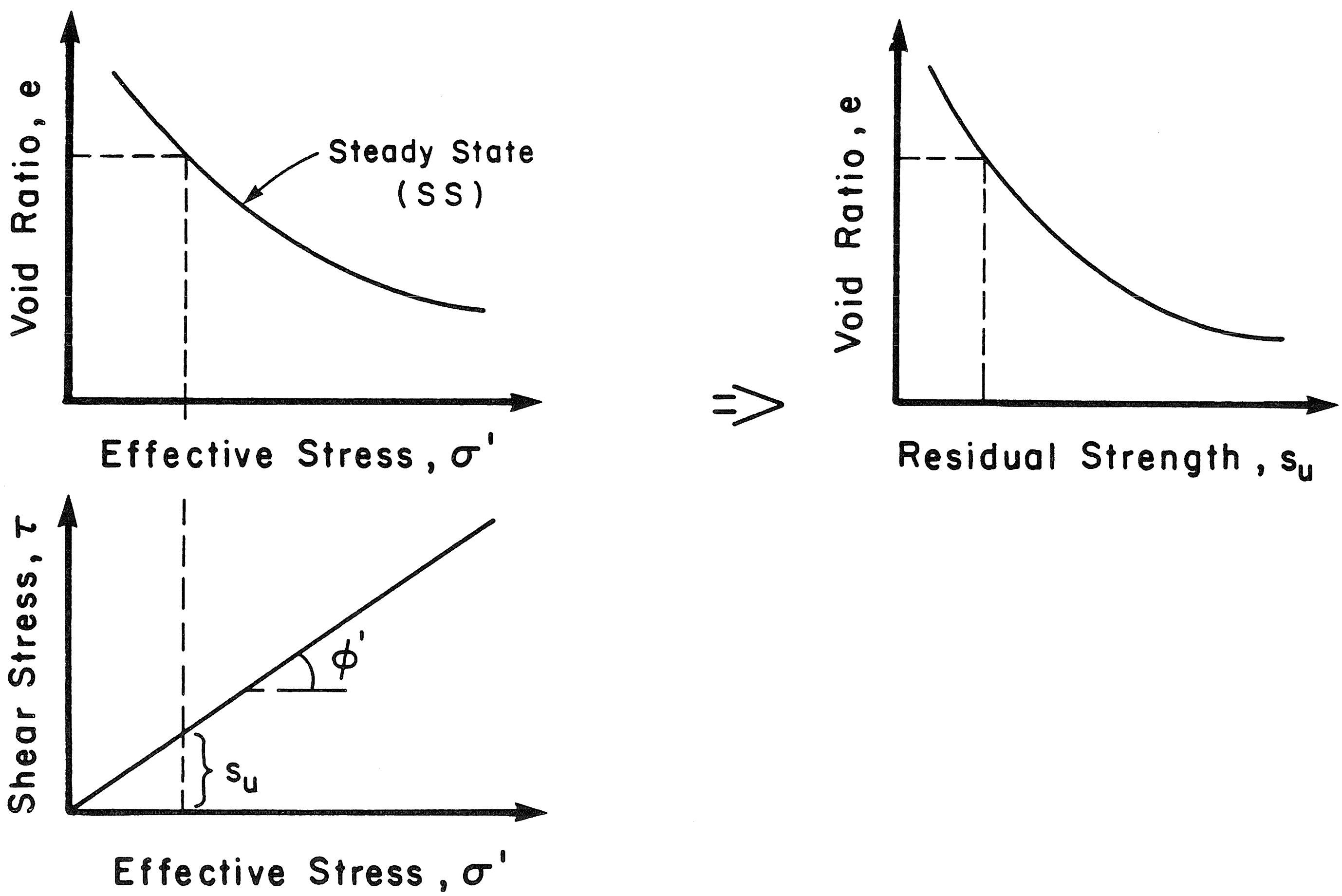


FIG.10: RESIDUAL STRENGTH AS A FUNCTION OF VOID RATIO FROM MONOTONIC LOADING OF SATURATED SANDS.

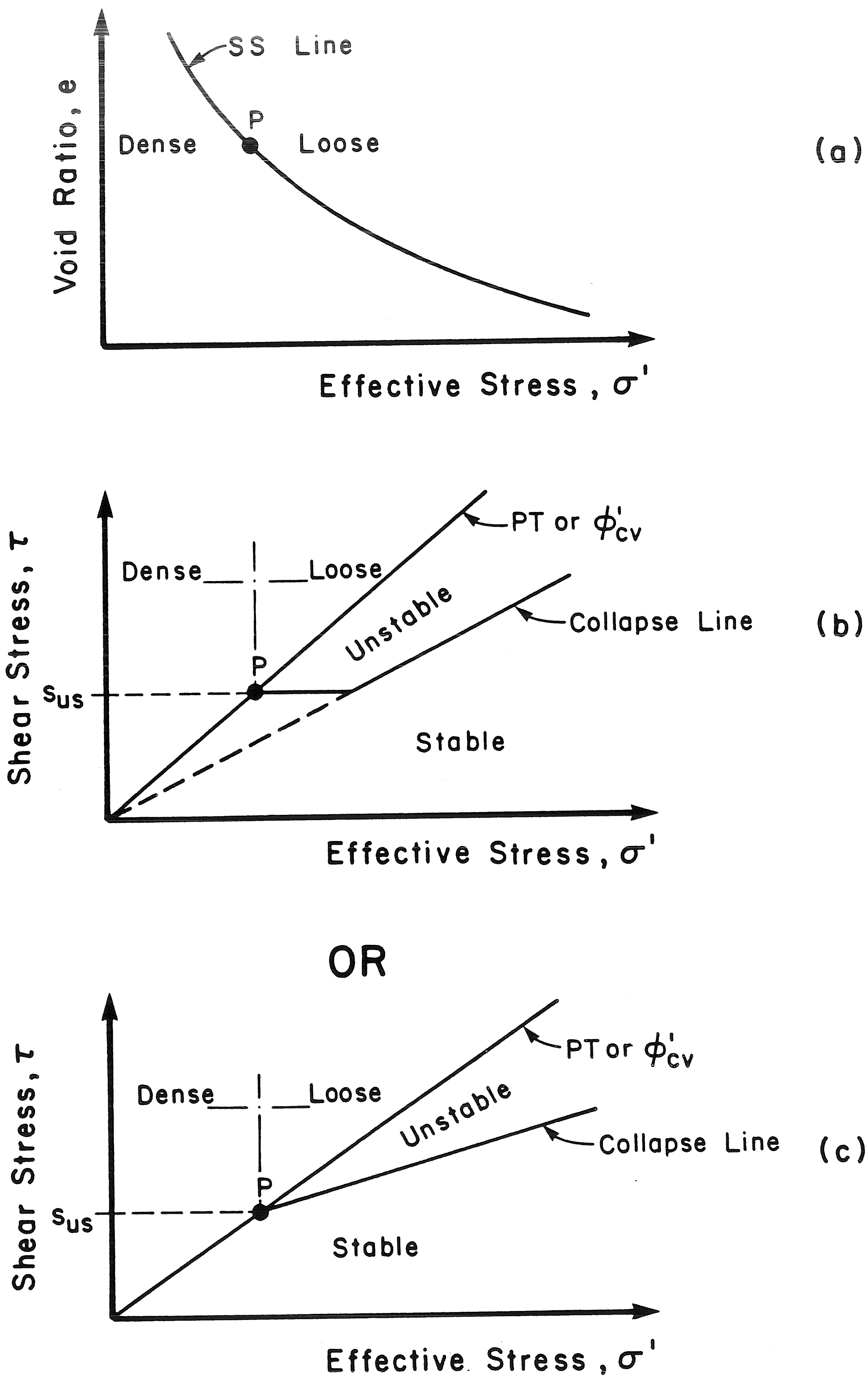
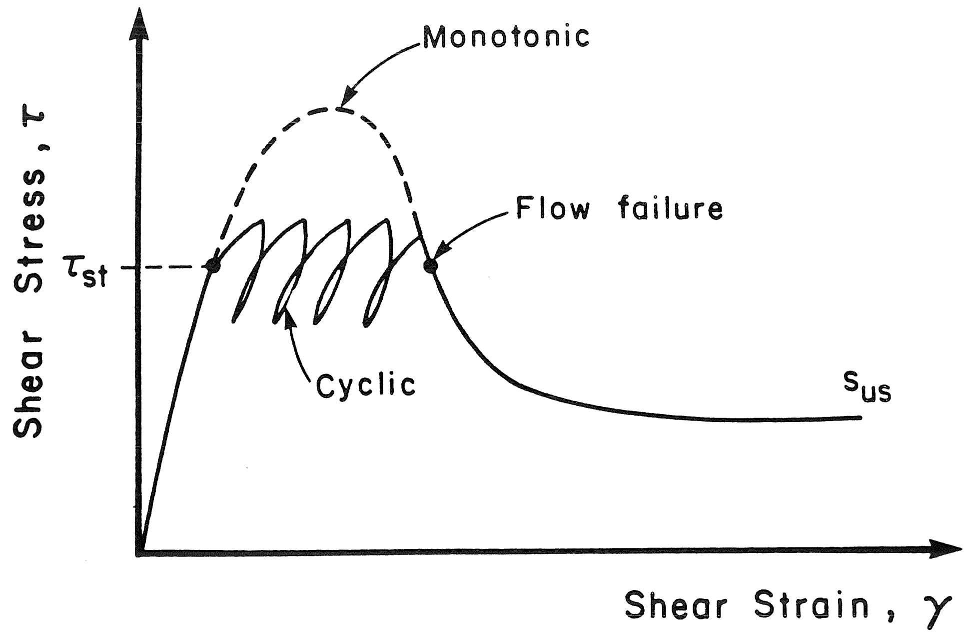
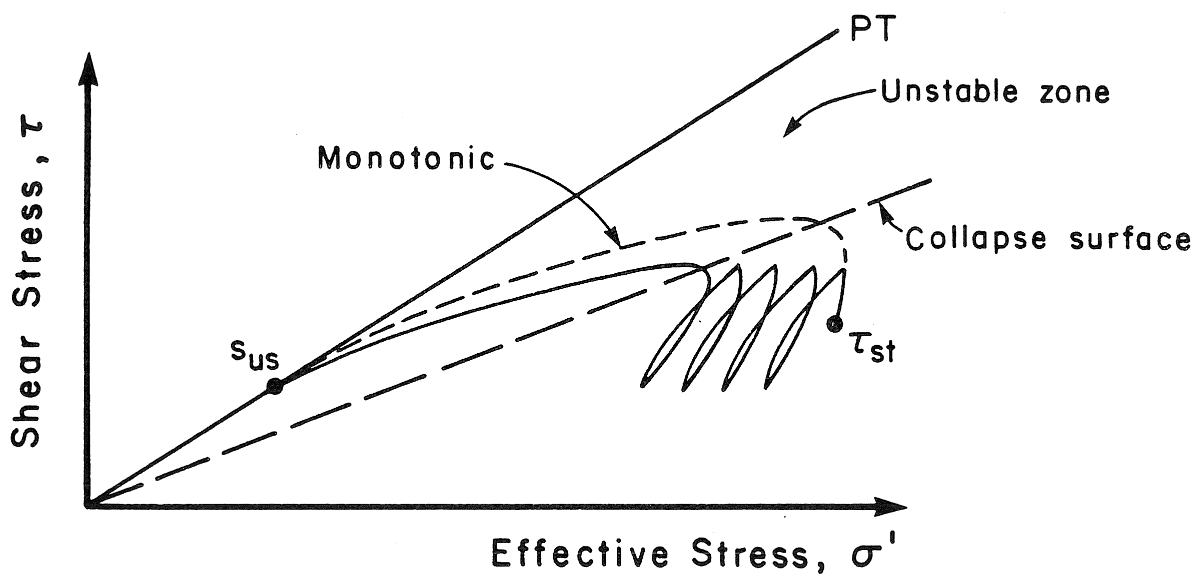


FIG.11: STABLE AND UNSTABLE ZONES FROM MONOTONIC TESTS ON SATURATED UNDRAINED SAND SAMPLES.



(a)



(b)

FIG. 12: RESPONSE OF LOOSE SATURATED SAND UNDER UNDRAINED CYCLIC LOADING.

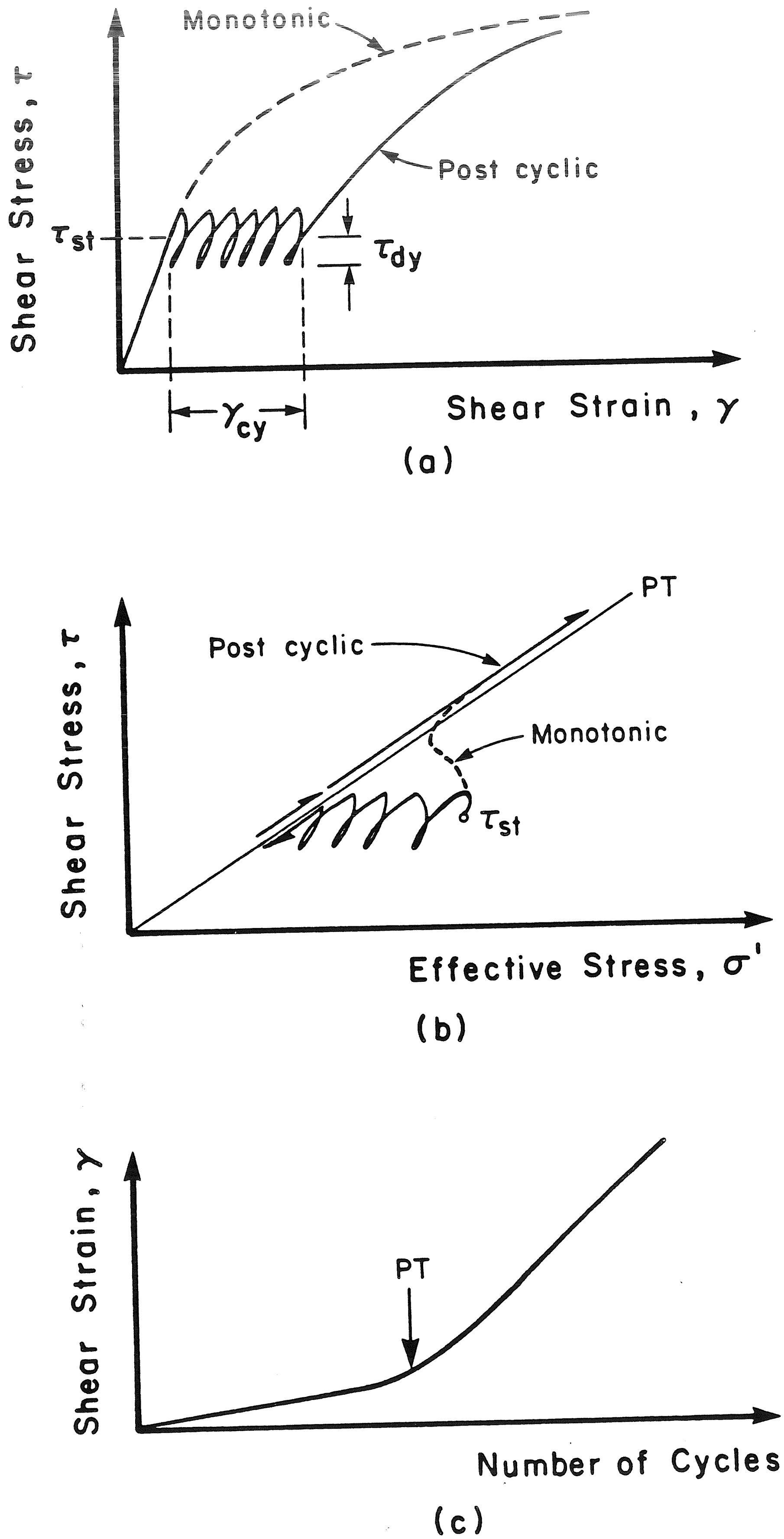


FIG.13: RESPONSE OF DENSE SATURATED SAND UNDER UNDRAINED CYCLIC LOADING WITH τ_{st}.

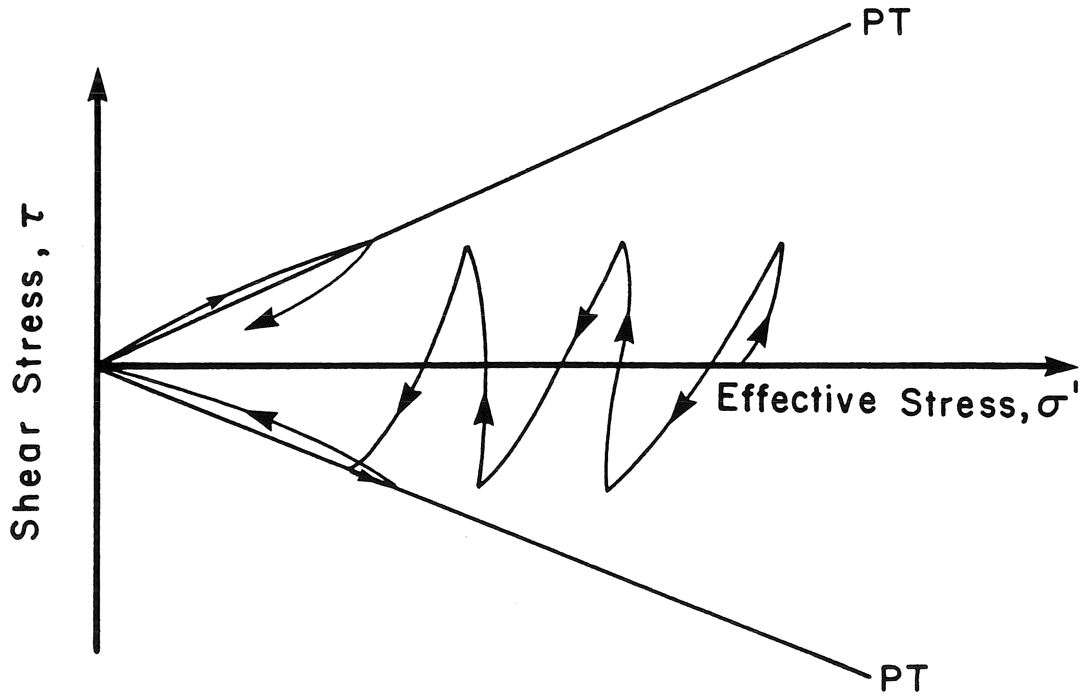
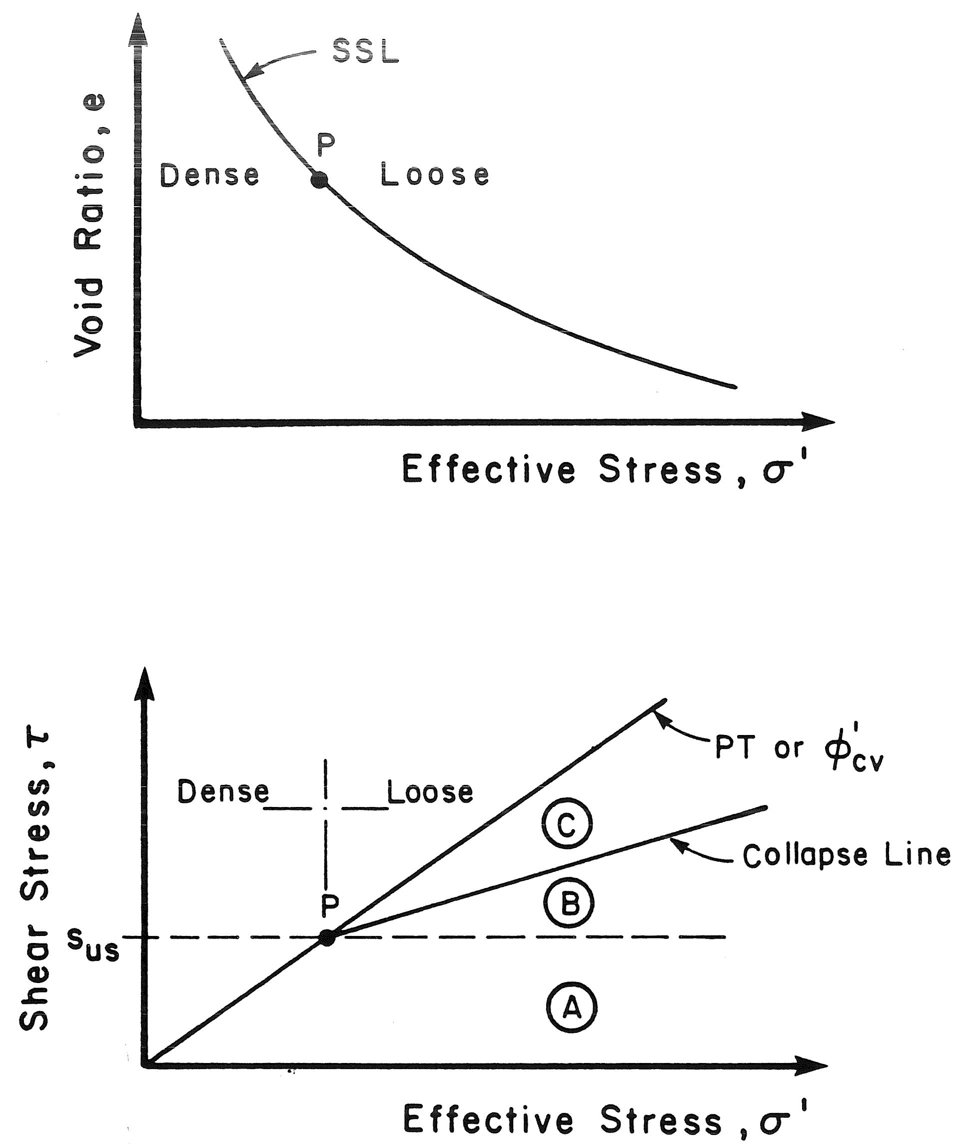


FIG. 14: RESPONSE OF DENSE SATURATED SAND UNDER UNDRAINED CYCLIC LOADING, $\tau_{st} = 0$.



Zone A $\tau_{st} < s_{us} \Rightarrow$ No Liquefaction

Zone B $\tau_{st} > s_{us} \Rightarrow$ Liquefaction can be triggered

Zone C $\tau_{st} \gg s_{us} \Rightarrow$ Spontaneous Liquefaction

FIG. 15: STATE DIAGRAM FOR SATURATED SAND.

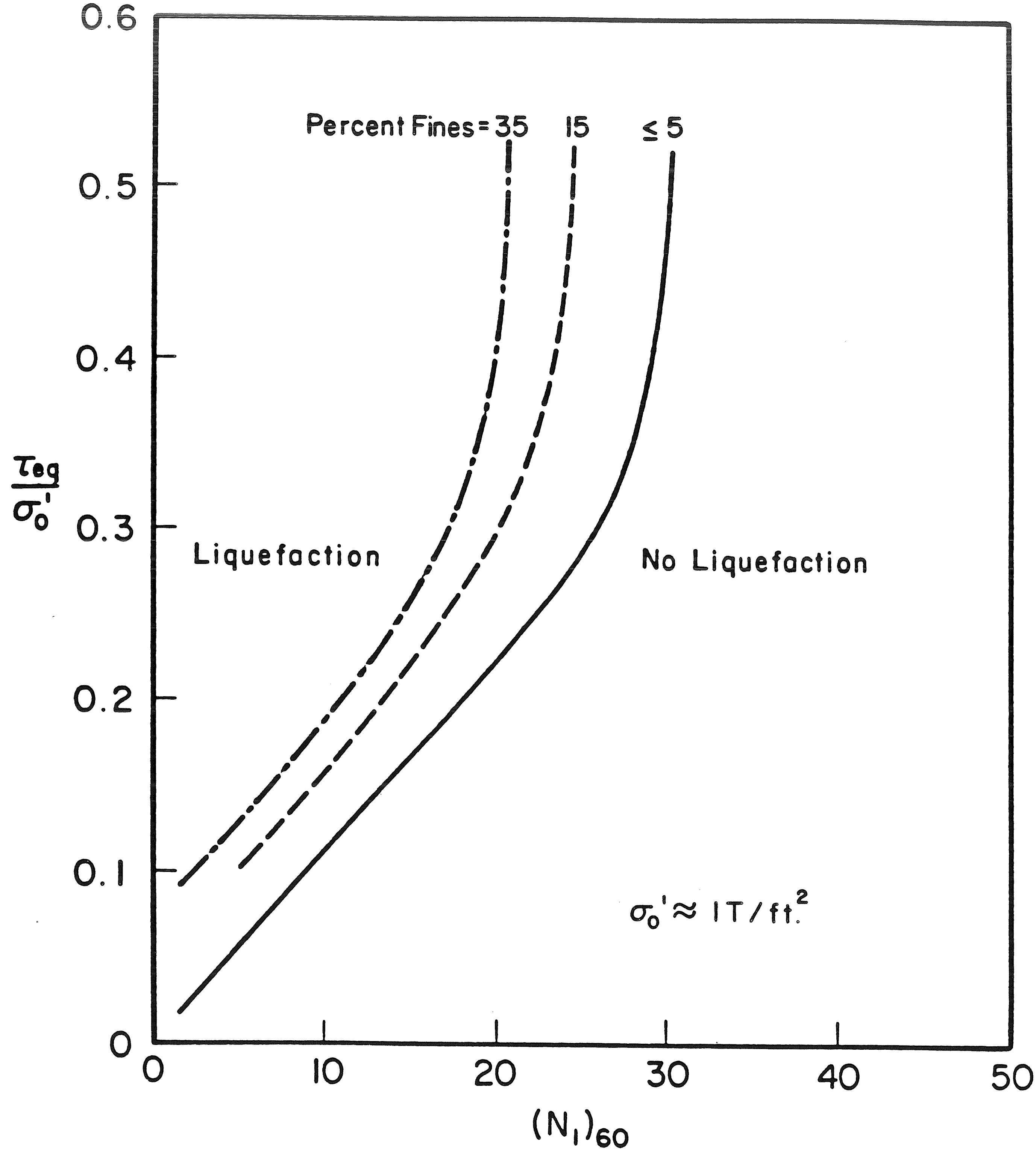


FIG. 16: RELATIONSHIPS BETWEEN STRESS RATIO CAUSING LIQUEFACTION AND N_1 – VALUES FOR SILTY SANDS FOR M = 7-1/2 EARTHQUAKES.
(Seed et al., 1984)

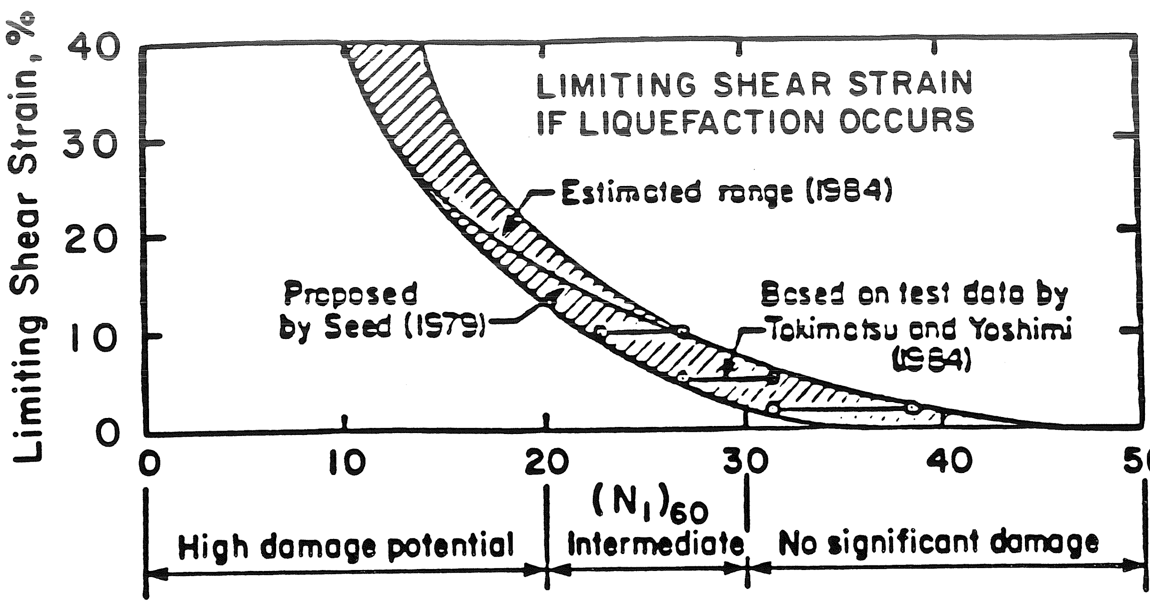


FIG. 17: SHEAR STRAINS AND LIKELY DAMAGE AS A FUNCTION OF N_1 IF LIQUEFACTION IS TRIGGERED.
(Seed et al., 1984)

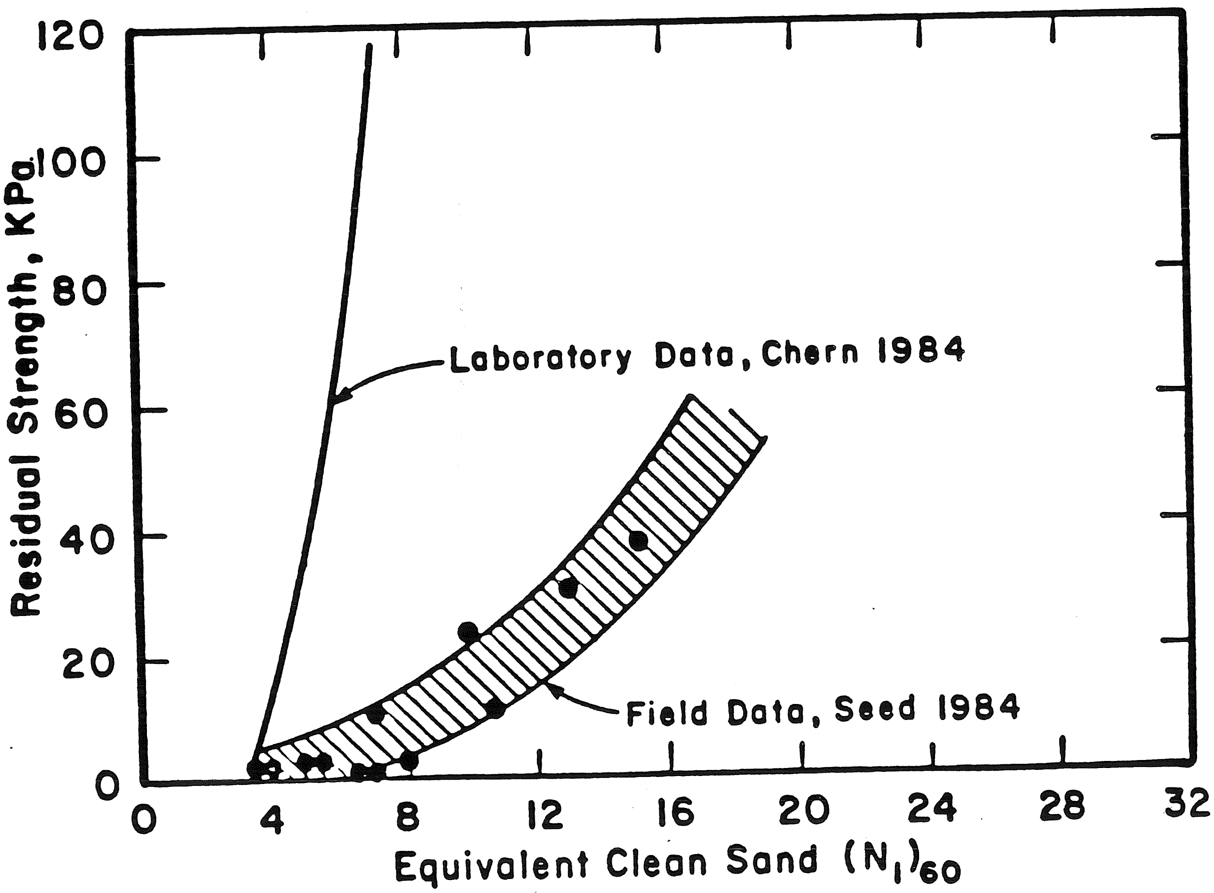
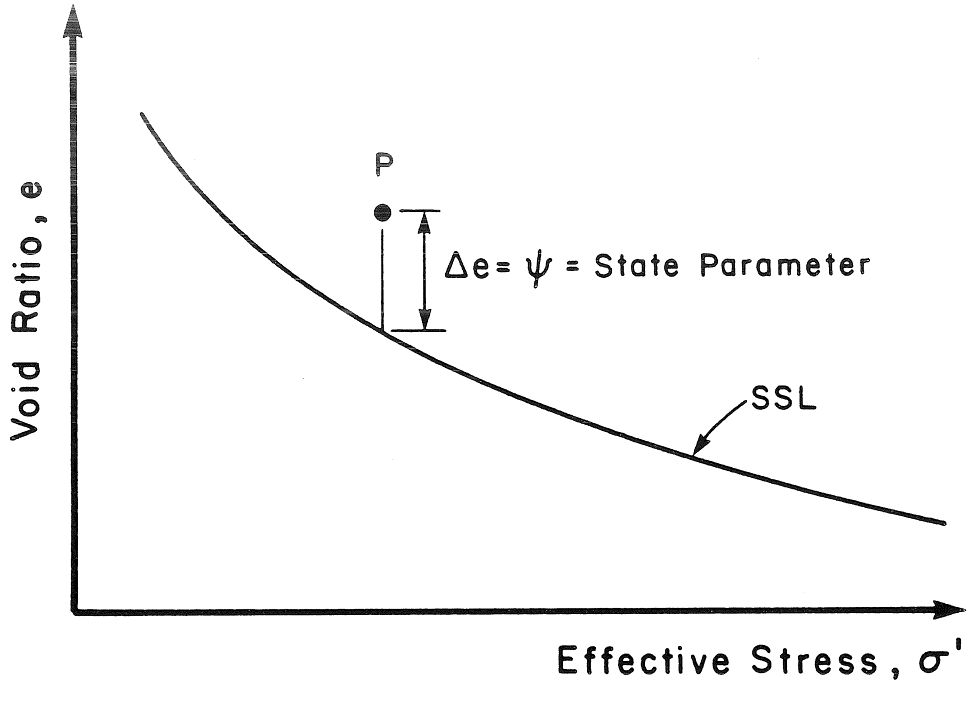
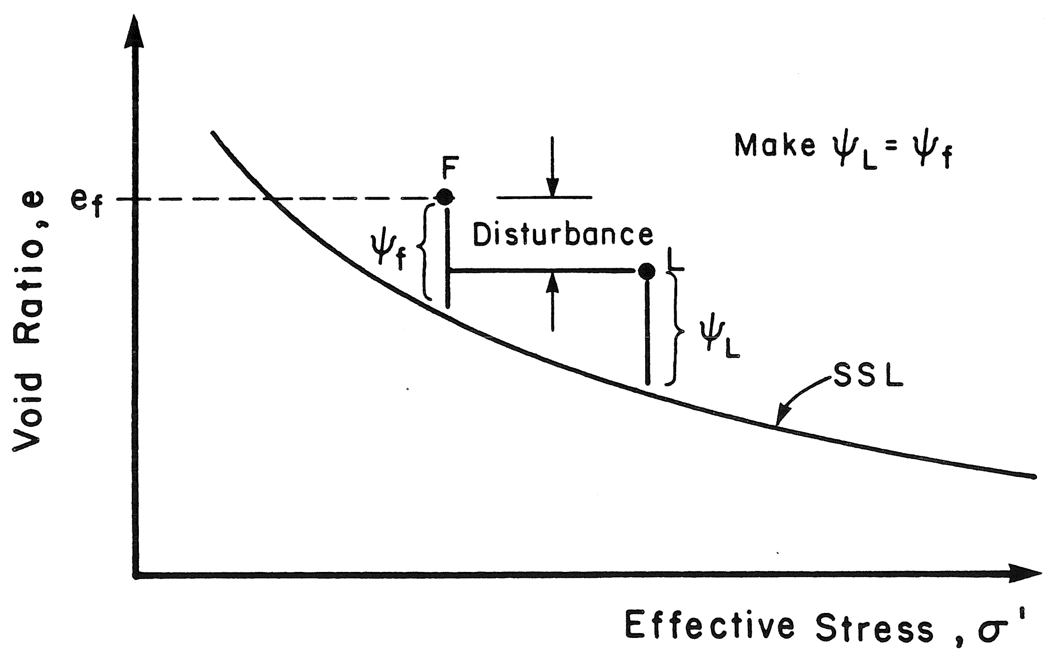


FIG. 18: TENTATIVE RELATIONSHIP BETWEEN RESIDUAL STRENGTH AND SPT N-VALUES FOR SANDS.



(a)

State Parameter



(b)

Correct Stress Level to keep same ψ

FIG.19: METHOD FOR CORRECTING LABORATORY SAMPLES TO ALLOW FOR SAMPLE DISTURBANCE BASED ON THE STATE PARAMETER.