

## **II. Ground Response Analysis & Evaluation of Liquefaction Potential**

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## 1. Ground Response Analysis

### 1.1 Introduction

#### Ground response analyses

- predict ground surface motions for development of design response spectra
- evaluate dynamic stresses and strains for evaluation of liquefaction hazards
- determine the earthquake-induced forces that can lead to instability of earth and earth-retaining structures.

### 1.2 One-Dimensional Ground Response Analysis

#### Assumption

- all boundaries are horizontal
- the response of a soil deposit is predominantly caused by SH-wave propagating vertically from the underlying bedrock.

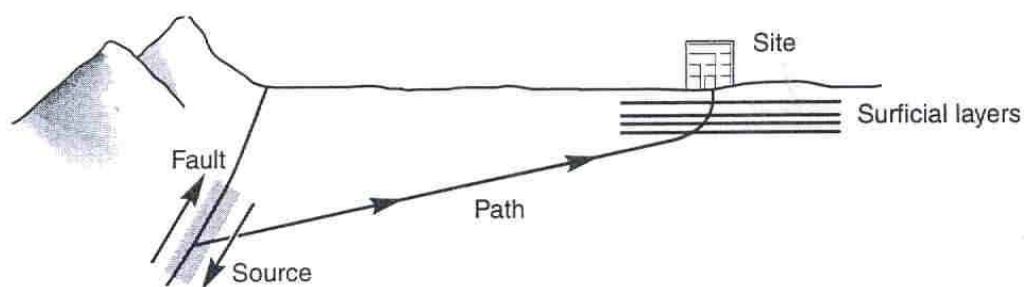


Figure 1.1 Refraction process that produces nearly vertical wave propagating near the ground surface.

free surface motion

The motion at the surface of a soil deposit

bedrock motion

The motion at the base of the soil deposit (also the top of bedrock)

rock outcropping motion

The motion at a location where bedrock is exposed at the ground surface

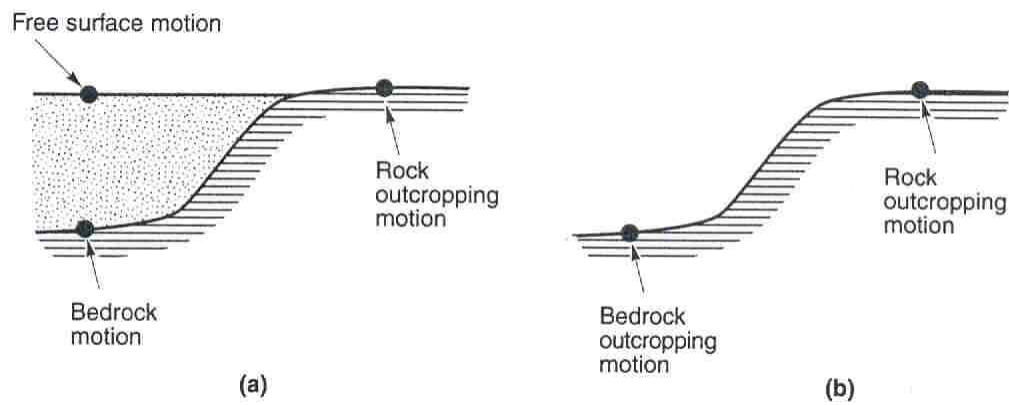


Figure 1.2 Ground response nomenclature: (a) soil overlying bedrock; (b) no soil overlying bedrock. Vertical scale is exaggerated.

### 1.2.1 Linear Approach

- An important class of techniques for ground response analysis is also based on the use of transfer function.
- For the ground response problem, transfer functions can be used to express various response parameters, such as displacement, velocity, acceleration, shear stress, and shear strain, to an input motion parameter such as bedrock acceleration.
- relies on superposition → limited to the analysis of linear systems
- A known time history of bedrock (input) motion is represented as a Fourier series, usually using the FFT
  - Each term in the Fourier series of the bedrock (input) motion is then multiplied by the transfer function
  - produce the Fourier series of the ground surface (output) motion
  - The ground surface (output) motion can then be expressed in the time domain using the inverse FFT.
- the transfer function determines how each frequency in the bedrock (input) motion is amplified, or deamplified, by the soil deposit.
- The key to the linear approach → the evaluation of transfer functions

Analysis Theory : One-dimensional harmonic shear wave propagation

Consider a soil deposit consisting of N horizontal layers where the Nth layer is bedrock. The wave equation is of the form given in equation (1.1).

$$\rho \frac{\partial^2 u}{\partial t^2} = G \frac{\partial^2 u}{\partial x^2} + \eta \frac{\partial^3 u}{\partial x^2 \partial t} \tag{1.1}$$

Where,  $u$ : shear strain,  $\rho$ : density,  $G$ : shear modulus,  $\eta$ : viscosity

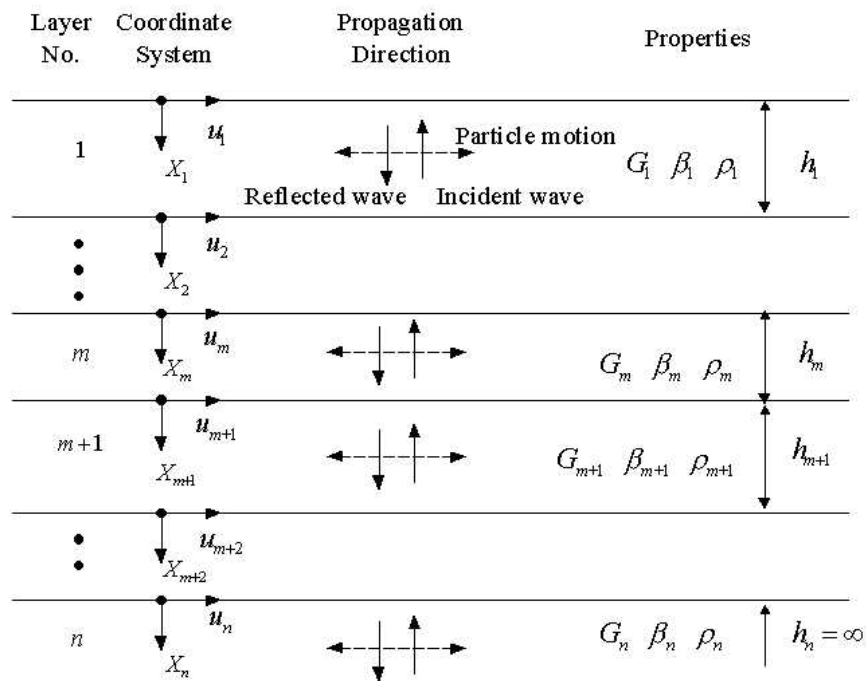


Figure 1.3 Nomenclature for layered soil deposit

The solution to the wave equation can be expressed in the form

$$u(x,t) = E e^{i(kx + \omega t)} + F e^{-i(kx - \omega t)} \tag{1.2}$$

Where E and F represent the amplitudes of waves traveling in the  $-x$ (upward) and  $+x$ (downward) directions, respectively. The shear stress is then given by the product

of the complex shear modulus,  $G^*$ , and the shear strain, so

$$\tau(x,t) = G \cdot \frac{\partial u}{\partial x} + \eta \frac{\partial^2 u}{\partial x \partial t} = G^* \cdot \frac{\partial u}{\partial x} \quad (1.3)$$

Where,  $G^* = G + i\omega\eta$ , if using  $\omega\eta = 2G\beta$ , then  $G^* = G(1 + 2i\beta)$ .

Introducing a local coordinate system,  $x$ , for each layer, the displacement at the top of a particular layer must be equal to the displacement at the top and bottom of layer  $m$  will be

$$u_m(x=0) = (E_m + F_m) e^{i\omega t} \quad (1.4)$$

$$u_m(x=h_m) = (E_m e^{ik_m h_m} + F_m e^{-ik_m h_m}) e^{i\omega t} \quad (1.5)$$

The shear stresses at the top and bottom of layer  $m$  are

$$\tau_m(x=0) = ik_m G_m^* (E_m - F_m) e^{i\omega t} \quad (1.6)$$

$$\tau_m(x=h_m) = ik_m G_m^* (E_m e^{ik_m h_m} - F_m e^{-ik_m h_m}) e^{i\omega t} \quad (1.7)$$

Since stresses must be continuous at layer boundaries, so

$$E_{m+1} - F_{m+1} = \frac{k_m G_m^*}{k_{m+1} G_{m+1}^*} (E_m e^{ik_m h_m} - F_m e^{-ik_m h_m}) \quad (1.8)$$

$$E_{m+1} + F_{m+1} = \frac{k_m G_m^*}{k_{m+1} G_{m+1}^*} (E_m e^{ik_m h_m} + F_m e^{-ik_m h_m}) \quad (1.9)$$

Using (1.8) and (1.9), the recursion formulas are,

$$E_{m+1} = \frac{1}{2} E_m (1 + \alpha_m) e^{ik_m h_m} + \frac{1}{2} F_m (1 - \alpha_m) e^{-ik_m h_m} \quad (1.10)$$

$$F_{m+1} = \frac{1}{2} E_m (1 - \alpha_m) e^{ik_m h_m} + \frac{1}{2} F_m (1 + \alpha_m) e^{-ik_m h_m} \quad (1.11)$$

Where  $\alpha_m^*$  is the complex impedance ratio at the boundary between layers m and m+1:

$$\alpha_m = \frac{k_m G_m^*}{k_{m+1} G_{m+1}^*} = \left( \frac{\rho_m G_m^*}{\rho_{m+1} G_{m+1}^*} \right)^{1/2} \quad (1.12)$$

At the ground surface, the shear stress must be equal to zero, which requires that  $E_1 = F_1$ . If the recursion formula of equation (1.10) & (1.11) are applied repeatedly for all layers from 1 to m, functions relating the amplitudes in layer m to those in layer 1 can be expressed by

$$E_m = e_m(\omega) E_1 \quad (1.13)$$

$$F_m = f_m(\omega) E_1 \quad (1.14)$$

The transfer function relating the displacement amplitude at layer i to that at layer j is given by

$$F_{ij}(\omega) = \frac{|u_i|}{|u_j|} = \frac{e_i(\omega) + f_i(\omega)}{e_j(\omega) + f_j(\omega)} \quad (1.15)$$

Equation (1.15) also describes the amplification of accelerations and velocities from layer i to layer j. Equation (1.15) indicates that the motion in any layer can be determined from the motion in any other layer. Hence if the motion at any one point in the soil profile is known, the motion at any other point can be contributed. This result allows a very useful operation called deconvolution to be performed.

### 1.2.2 Equivalent Linear Approximation of Nonlinear Response

- nonlinearity of soil behavior → linear approach must be modified to provide reasonable estimates of ground response for practical problems of interest.
- The equivalent linear shear modulus,  $G$ , is generally taken as a secant shear modulus and the equivalent linear damping ratio,  $\xi$ , as the damping ratio that produces the same energy loss in a single cycle as the actual hysteresis loop.
- is common to characterize the strain level of the transient record in terms of an effective shear strain which has been empirically found to vary between about 50 and 70% of the maximum shear strain.
- the effective shear strain is often taken as 65% of the peak strain.

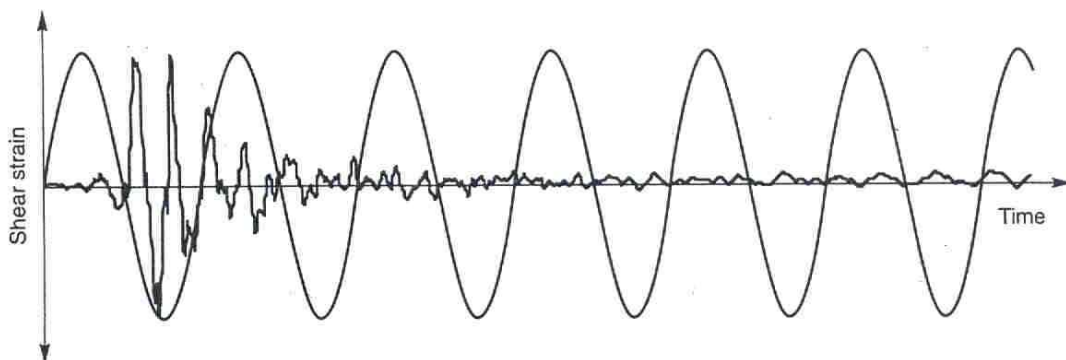


Figure 1.4 Two shear strain time histories with identical peak shear strains.

For the transient motion of an actual earthquake, the effective shear strain is usually taken as 65% of the peak strain.



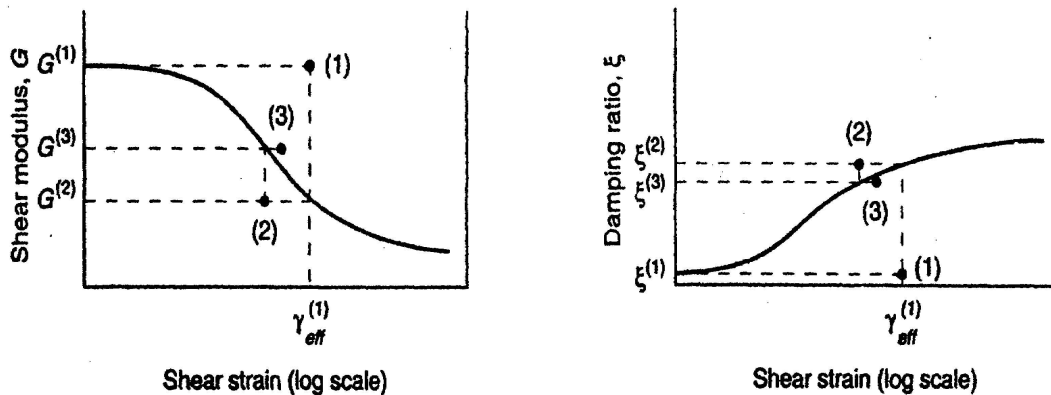


Figure 1.5 Iteration toward strain-compatible shear modulus and damping ratio in equivalent linear analysis.

- computed strain level depends on the values of the equivalent linear properties → an iterative procedure is required to ensure that the properties used in the analysis are compatible with the computed strain levels in all layers.

– the iterative procedure (Referring to Figure 1.5)

1. Initial estimates of  $G$  and  $\xi$  are made for each layer. The initially estimated values usually correspond to the same strain level; the low-strain values are often used for the initial estimate.
2. The estimated  $G$  and  $\xi$  values are used to compute the ground response, including time histories of shear strain for each layer.
3. The effective shear strain in each layer is determined from the maximum shear strain in the computed shear strain time history. For layer  $j$

$$\gamma_{eff}^{(i)} = R_{\gamma} \gamma_{maxj}^{(i)}$$

where the superscript refers to the iteration number and  $R_{\gamma}$  is the ratio of the

effective shear strain to maximum shear strain.  $R_\gamma$  depends on earthquake magnitude (Idriss and Sun, 1992) and can be estimated from

$$R_\gamma = \frac{M-1}{10}$$

4. From this effective shear strain, new equivalent linear values,  $G^{(i+1)}$  and  $\xi^{(i+1)}$  are chosen for the next iteration.
  5. Steps 2 to 4 are repeated until differences between the computed shear modulus and damping ratio values in two successive iterations fall below some predetermined value in all layers. Although convergence is not absolutely guaranteed, differences of less than 5 to 10% are usually achieved in three to five iterations (Schnabel et al., 1972).
- Even though the process of iteration toward strain-compatible soil properties allows nonlinear soil behavior to be approximated, it is important to remember that the complex response method is still a linear method of analysis.
  - The strain-compatible soil properties are constant throughout the duration of the earthquake, regardless of whether the strains at a particular time are small or large.
  - The method is incapable of representing the changes in soil stiffness that actually occur during the earthquake.
  - The equivalent linear approach to one-dimensional ground response analysis of layered sites has been coded into a widely used computer program called SHAKE(Schnabel et al., 1972).

### 1.3 Two-Dimensional Ground Response Analysis

- One-dimensional ground response analyses are based on the assumption that all boundaries are horizontal and that the response of a soil deposit is predominantly caused by SH-wave propagating vertically from the underlying bedrock.
- Two- and three-dimensional dynamic response and soil-structure interaction problems are most commonly solved using dynamic finite-element analyses.

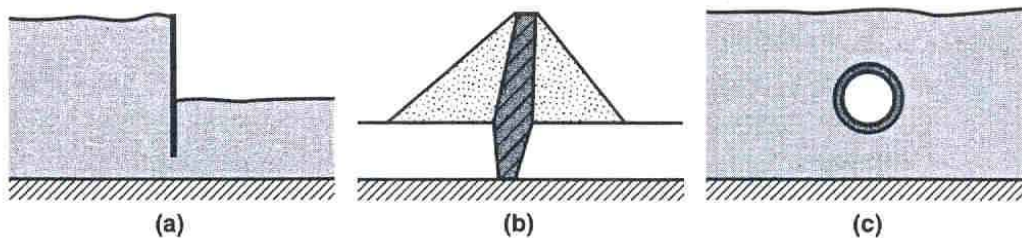


Figure 1.6 Examples of common problems typically analyzed by two-dimensional plane strain dynamic response analyses: (a) cantilever retaining wall; (b) earth dam; (c) tunnel.

## 2. Evaluation of Liquefaction Potential

### 2.1 Introduction

- The loss of strength may take place in sandy soils due to an increase in pore pressure. This phenomenon, termed *liquefaction*, can occur in loose and saturated sands. The increase in pore pressure causes a reduction in the shear strength behaves like a viscous fluid.
  
- By comparing the shear stresses induced by the earthquake with those required to cause liquefaction, determine whether any zone exists within the deposit where liquefaction can be expected to occur (induced stresses exceed those causing failure).

### 2.2 Criteria of Liquefaction Potential

- Data needed to evaluation of liquefaction potential

- 1) geological and topographical data
- 2) grain-size distribution, initial relative density, ground water level
- 3) shear modulus and damping ratio of each layer with strain level
- 4) results of field test (ex, SPT) and laboratory test (ex, cyclic shear test)
- 5) magnitude of design earthquake(peak ground acceleration and duration)

- Omission of evaluation of liquefaction potential

- 1) Level 2 structure in Zone II region
- 2) ground below water level
- 3)  $N \geq 20$

- 4)  $PI \geq 10$  and clay content  $\geq 20\%$
- 5) fines content  $\geq 35\%$
- 6) relative density  $\geq 80\%$
- 7) site classified as SA ~ SD

- magnitude of design earthquake : M 6.5 (zone I, zone II]
  
- Design earthquake motion should include at least 3 motions, which consist of real earthquake motion of long period, real earthquake motion of short period, and artificial earthquake motion, which satisfies design response spectrum (Seismic Design Guidelines of Port and Harbor Structures in Korea, 1999).
  
- in case of Level I structure, ground response analysis must be carried out. The resistance shear stress must be determined by the result of cyclic triaxial test.

## 2.3 Evaluation of Liquefaction Potential

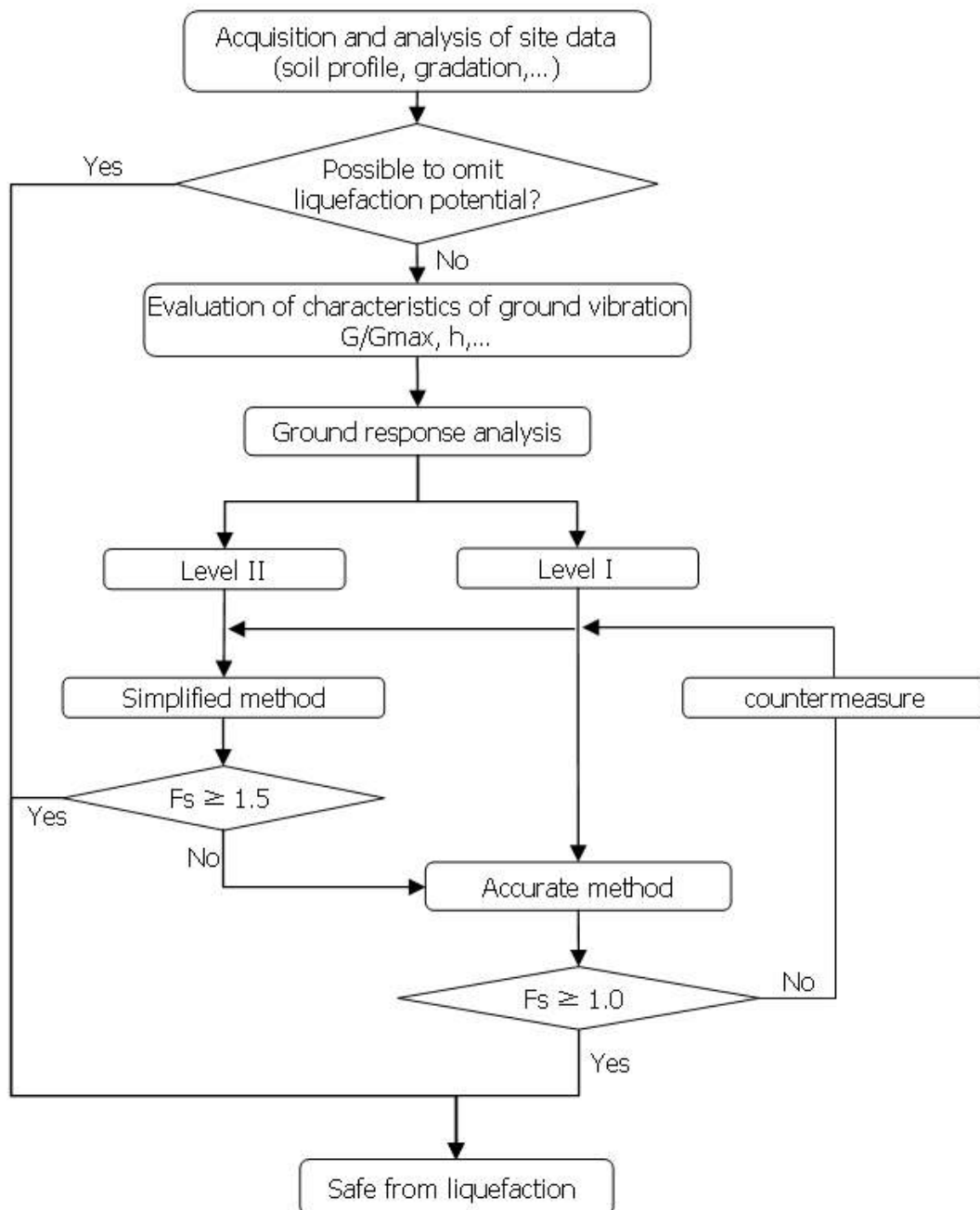


Figure 2.1 Procedure for evaluation of liquefaction potential

### 2.3.1 Simplified Method for Evaluation of Liquefaction Potential

– evaluation of liquefaction potential based on Seed & Idriss method(1971)

1) Safety factor of liquefaction potential is defined as ratio of resistance shear stress,  $\tau_l$  to shear stress driven by earthquake.

2) Cyclic shear stress driven by earthquake

$$\frac{\tau_l}{\sigma_v} = 0.65 \left( \frac{a_{\max}}{g} \right) \left( \frac{\sigma_v}{\sigma_v'} \right)$$

where,  $a_{\max}$  : maximum ground acceleration (by ground response analysis)

$g$  : acceleration of gravity

$\sigma_v$  : total overburden pressure

$\sigma_v'$  : effective overburden pressure

3) resistant shear stress can be obtained by 'N' value

① determine  $C_N$  (overburden correction factor)

$$C_N = \left( \frac{10}{\sigma_v'} \right)^{0.5} \quad ( C_N \leq 2 \text{ (Liao and Whitman, 1986), } \sigma_v' \text{ in tf/m}^2)$$

② determine corrected N value

$$N_i = N \cdot C_N$$

③ determine resistant cyclic stress ratio using  $N_i$  and chart (seed et al., 1975)

4) factor of safety against liquefaction, expressed as

$$F = \frac{\tau_l / \sigma_v'}{\tau_d / \sigma_v'}$$

5) Evaluation of liquefaction potential

$F \geq 1.5$  : not liquefied

$F \leq 1.5$  : liquefied, accurate evaluation method is needed

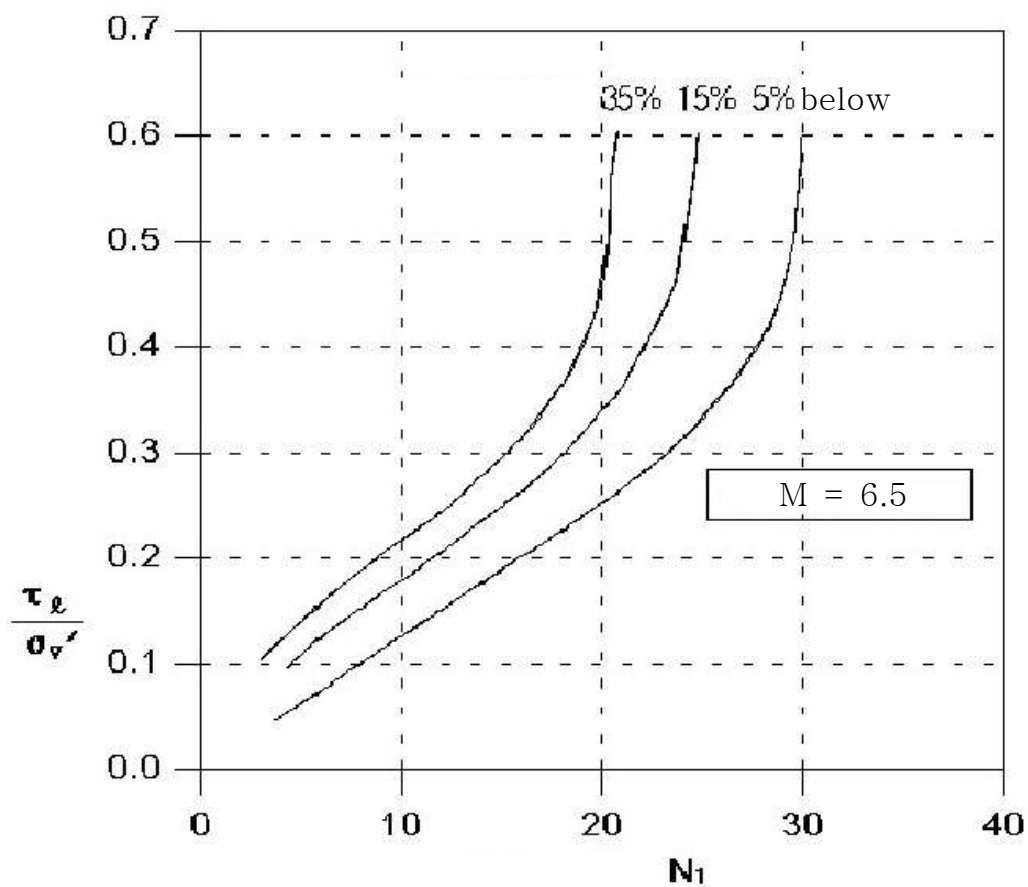


Figure 2.2 Relationship between cyclic stress ratios causing liquefaction and  $N_1$  value in M=6.5 earthquakes.



2.3.2 Accurate Method for Evaluation of Liquefaction Potential

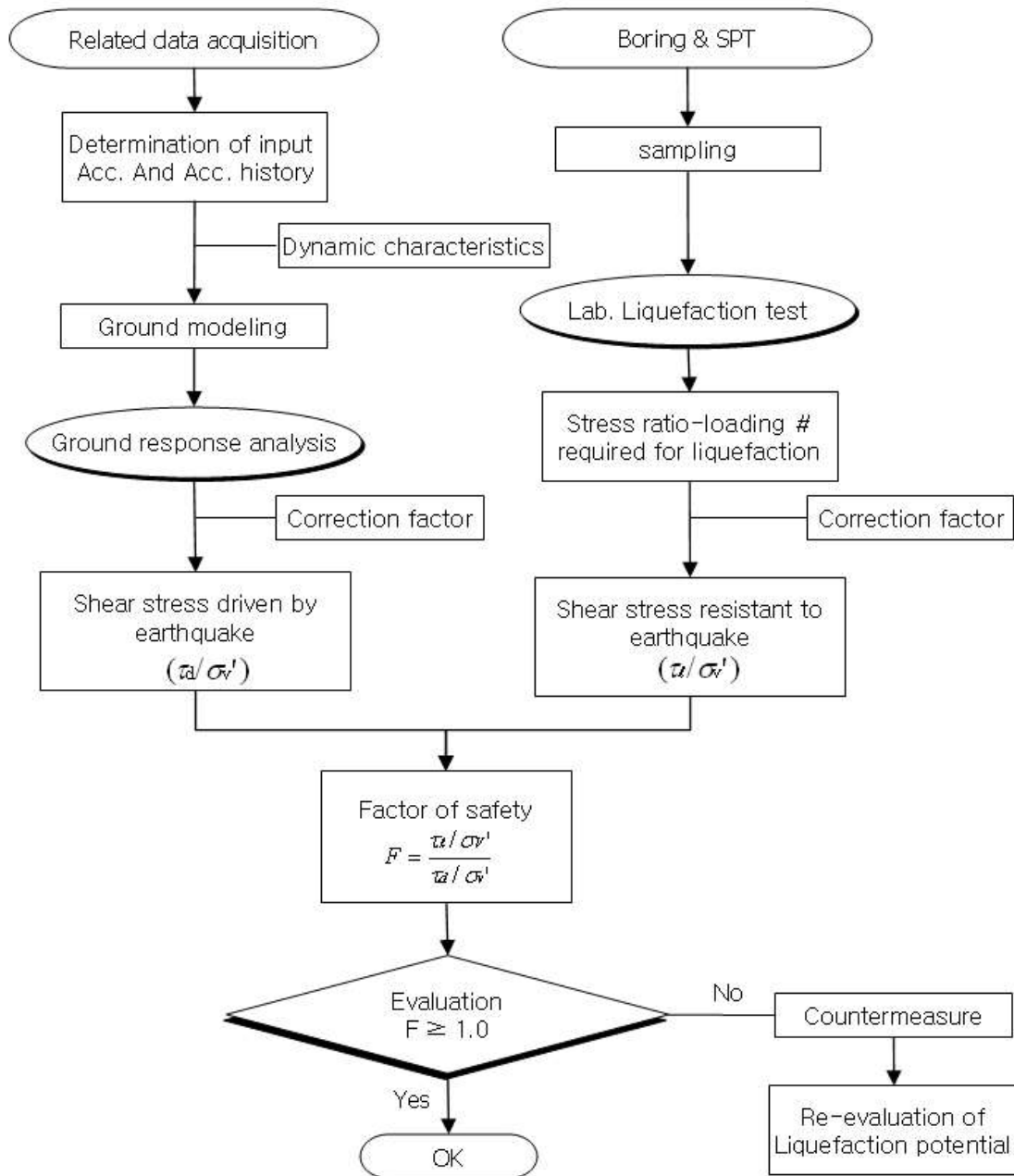


Figure 2.3 procedure for evaluation of liquefaction potential by accurate method

- The maximum cyclic **shear stress driven by earthquake** should be obtained by **ground response analysis** and the **resistant cyclic shear stress** should be obtained by the results of **cyclic triaxial test**.
- The characteristic curve for liquefaction resistant shear stress can be obtained by results of cyclic triaxial test. The shear stress resistant to liquefaction with depth can be evaluated by the characteristic curve.
- Evaluation of liquefaction potential

$F \geq 1.0$  : not liquefied

$F \leq 1.0$  : liquefied, countermeasure is needed

### 3. Application

#### 3.1 Evaluation of Liquefaction Potential for site of water facilities

- damage of water pipelines due to liquefaction



##### *3.1.1 Boring and field test for evaluation of Dynamic properties*

- boring and sampling



- field test

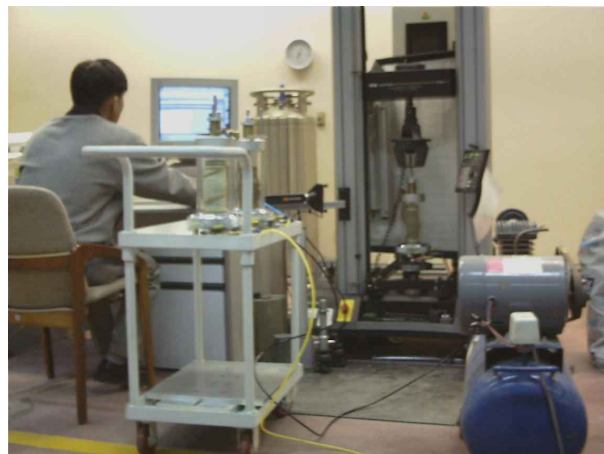


density investigation



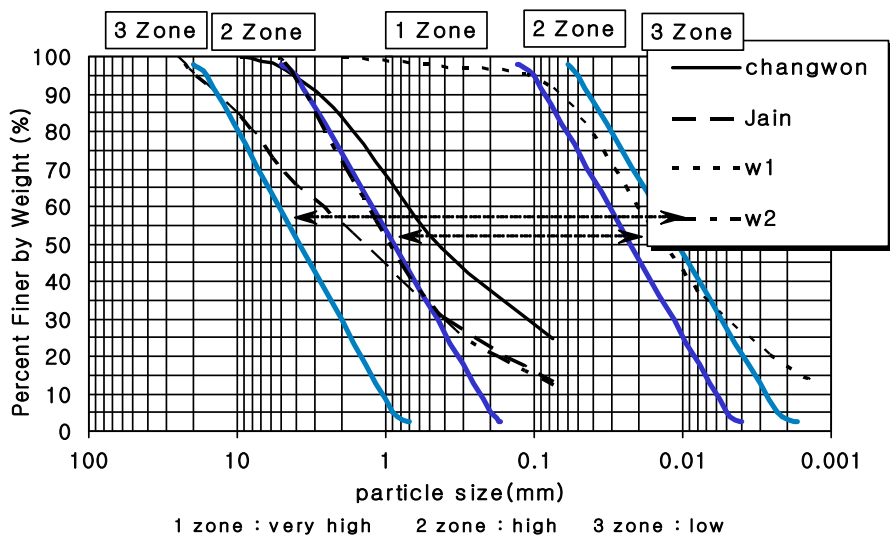
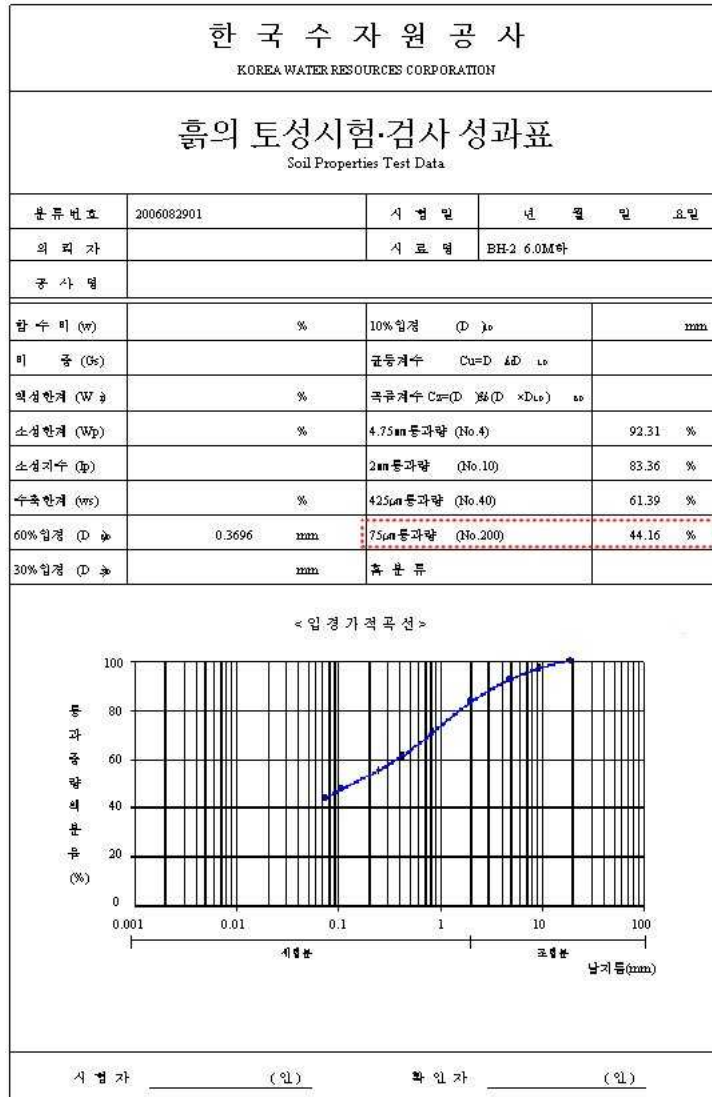
Suspension logging

- Laboratory test (cyclic triaxial test and resonance column test.....)





3) check - gradation characteristics

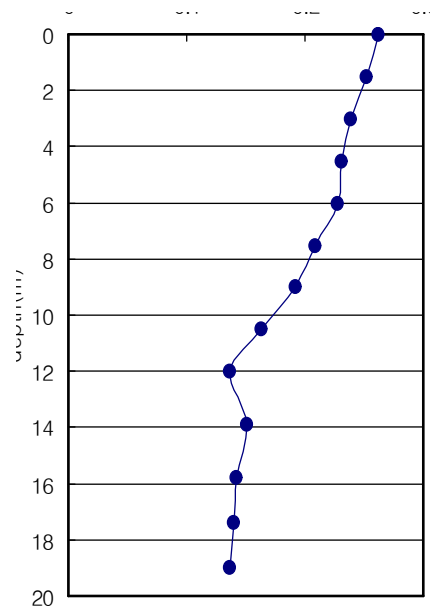


- Evaluation of liquefaction potential

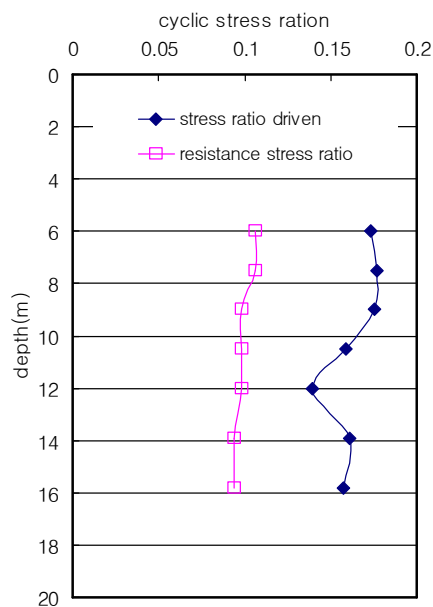
Site	Evaluation method			Evaluation
	preliminary	Simple	Accurate	
<b>Changwon</b>	○(O.K)	Not necceary	Not necceary	<b>Not liquefied</b>
<b>Pohang</b>	○(O.K)	Not necceary	Not necceary	<b>Not liquefied</b>
<b>Kumho</b>	○(O.K)	Not necceary	Not necceary	<b>Not liquefied</b>
<b>Jain</b>	○(O.K)	Not necceary	Not necceary	<b>Not liquefied</b>
<b>Ulsan</b>	○(O.K)	Not necceary	Not necceary	<b>Not liquefied</b>

3.1.3 Evaluation of liquefaction potential for water pipeline sites

- check whether the evaluation process can be omitted  
necessary to evaluation of liquefaction potential from soil profiles
- application of simplified method
  - 1) ground response analysis

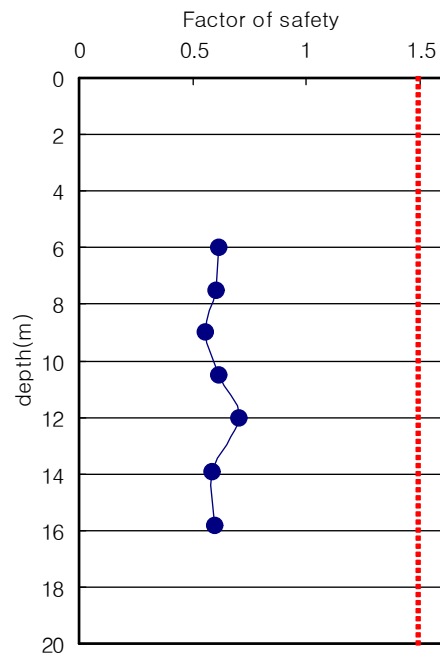


- 2) Determine cyclic stress ratio



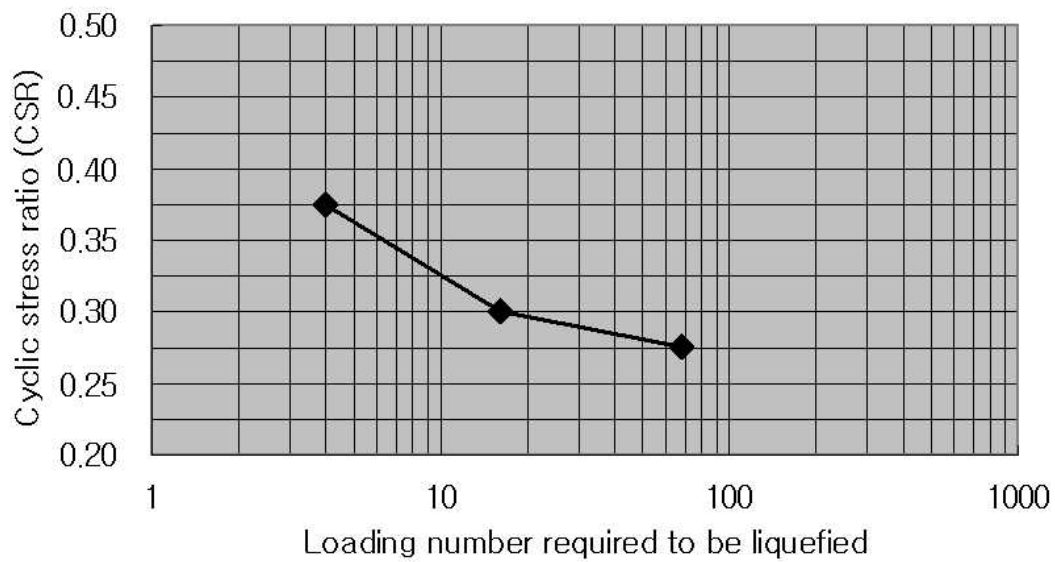


3) Factor of safety by simplified method

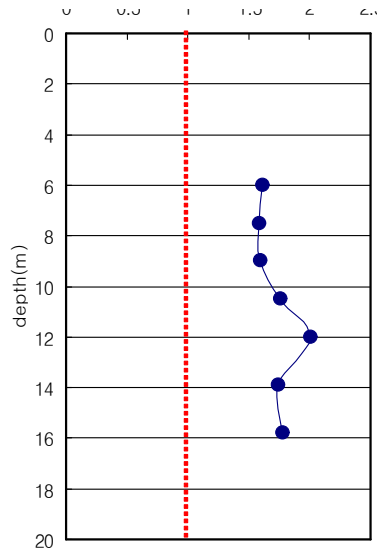


- application of accurate method

1) conduction of cyclic triaxial tests



2) Factor of safety by accurate method



- Final Evaluation of liquefaction potential

site	Evaluation method			Final Evaluation
	preliminary	Simple	Accurate	
Changwon	○(suspicious)	○(N.G)	○(O.K)	<b>Not liquefied</b>
Pohang	○(O.K)	Not necessary	Not necessary	<b>Not liquefied</b>
Kumho	○(O.K)	Not necessary	Not necessary	<b>Not liquefied</b>
Ulsan	○ (suspicious)	○(O.K)	Not necessary	<b>Not liquefied</b>
Jain	○ (suspicious)	○(O.K)	Not necessary	<b>Not liquefied</b>