Presentation Outline

1. Seismic Soil Liquefaction Explained
2. Presentation of the Software SOILLIQ
3. Illustrative Applications using SOILLIQ

Foundation Failures in Residential Buildings (1964 Niigata, Japan Earthquake)

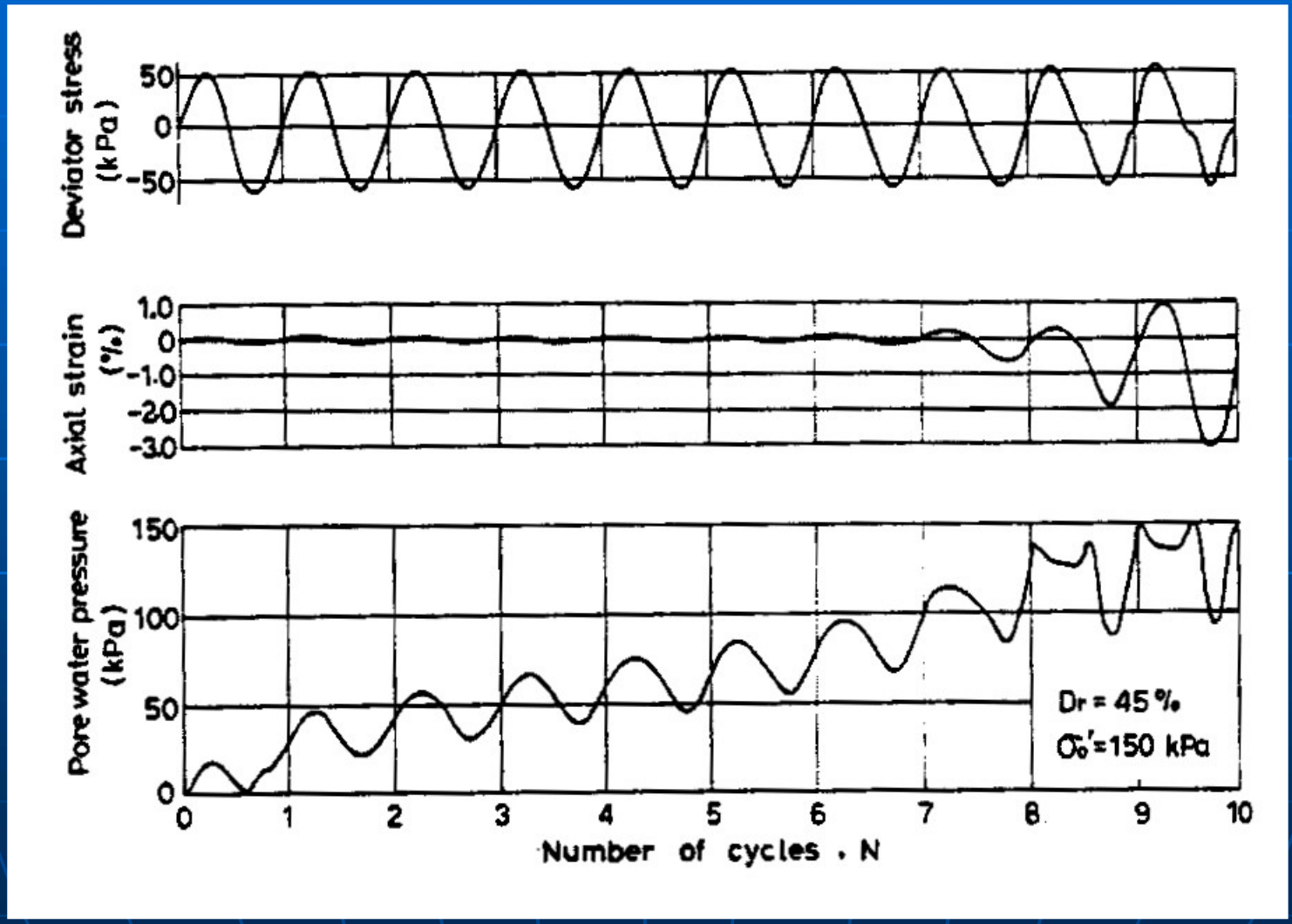


What Is Soil Liquefaction?

- During dynamic loading (as in the case of an earthquake) strength and stiffness of soil can be significantly reduced
- Eventually the soil begins to behave like a liquid rather than solid (liquefaction).
- Liquefaction typically occurs in saturated loose sands, which tend to contract in volume under dynamic loading.
- Earthquake shaking can cause the pore pressure, which is initially low, increase to the point where the soil particles can readily move with respect to each other (zero effective stress case).

$$\sigma' = \sigma - u \quad \Rightarrow \quad \tau = c' + \sigma' \tan \varphi'$$

Typical Soil Liquefaction during Dynamic Loading Tests on Saturated Sand (Ishihara, 1996)



Soil Liquefaction Related Phenomena

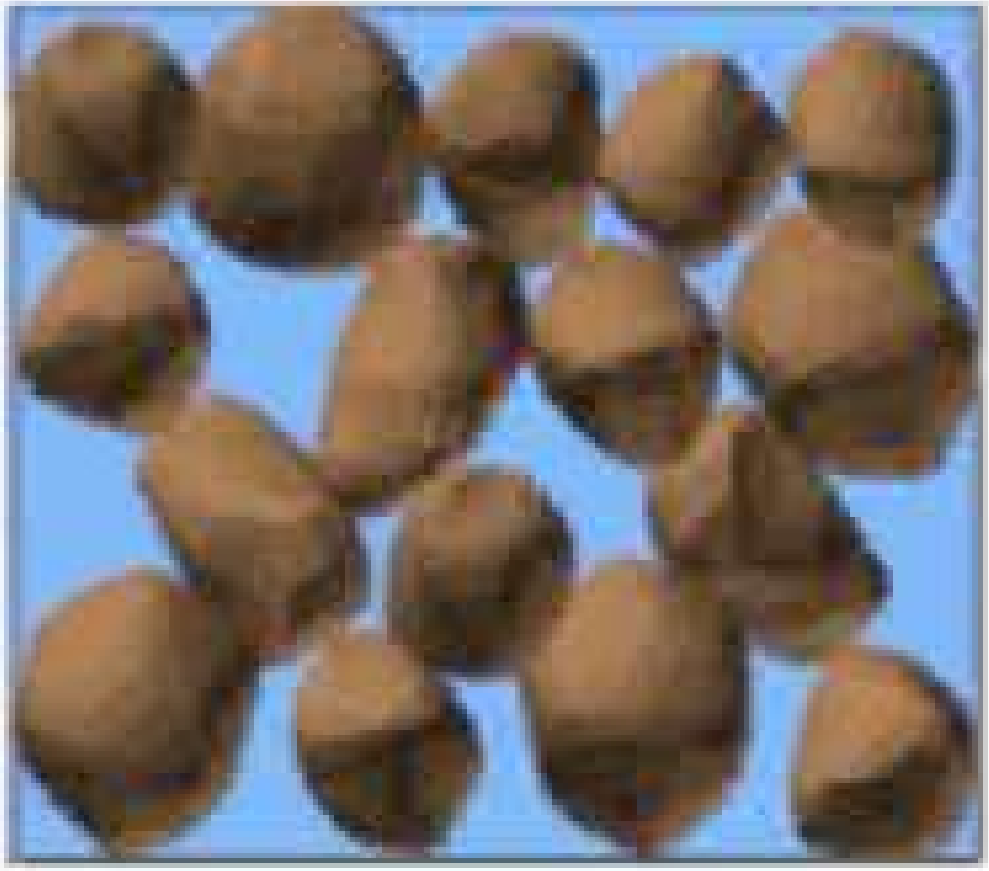
- Sand boils on the ground surface (solid evidence of liquefaction occurrence)
- Ground settlement
- Lateral spreading of slopes
- Ground oscillations (waving motion of the ground)
- Slope failures
- Damage in various forms on structures

Soil liquefaction related phenomena have been responsible for tremendous amounts of damage in historical earthquakes around the world.

Associated Adverse Effects on Structures

- Bearing capacity loss of foundations
- Distress (and hence damage) to superstructure due to differential settlements
- Increased pressures on retaining walls
- Floating of buried objects (like pipes and tanks)

Soil Skeleton during Liquefaction



Effects of Liquefaction on Surface Structures



Regions of man-made landfill fared poorly in the 1906 San Francisco where damage from the 1906 earthquake was associated with liquefaction and ground failure (adopted from web)



Many regions of man-made landfill liquefied in the 1989 Loma Prieta earthquake. The Marina district, a shallow bay filled in after the 1906 earthquake, suffered some of the worst damage in the 1989 earthquake (USGS photo)



Increased water pressure can also trigger landslides and cause the collapse of dams. Lower San Fernando dam suffered an underwater slide during the San Fernando earthquake, 1971 (adopted from web)



Liquefaction induced damages after 1964 Alaska Earthquake in Anchorage.





Liquefaction induced damages after 1995 Hyogo-Ken Nanbu Earthquake at Kobe (Japan).





Vehicles stuck in liquefaction sink holes during 2011 Christchurch Earthquake in New Zealand



Foundation Bearing Failures (Adapazari, Turkey, 17 August 1999 Izmit Earthquake)



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Liquefaction Induced Excessive Settlement of a Hotel Building (Southwest of Adapazari)



Liquefaction Induced Lateral Spreading (North of Van)





Sand Boilings (North of Van)





Factors Governing Liquefaction

- Earthquake intensity and duration
- Presence of groundwater
- Soil type
- Relative density
- Particle size gradation
- In-situ confining pressures
- Aging and cementation (binding of particles)

Assessment of Liquefaction Triggering

1. Through correlations with field penetration testing (based on the simplified method proposed by Seed et al., 1975)

2. Laboratory testing based methods

Field Penetration Testing Approach

Factor of safety against liquefaction:

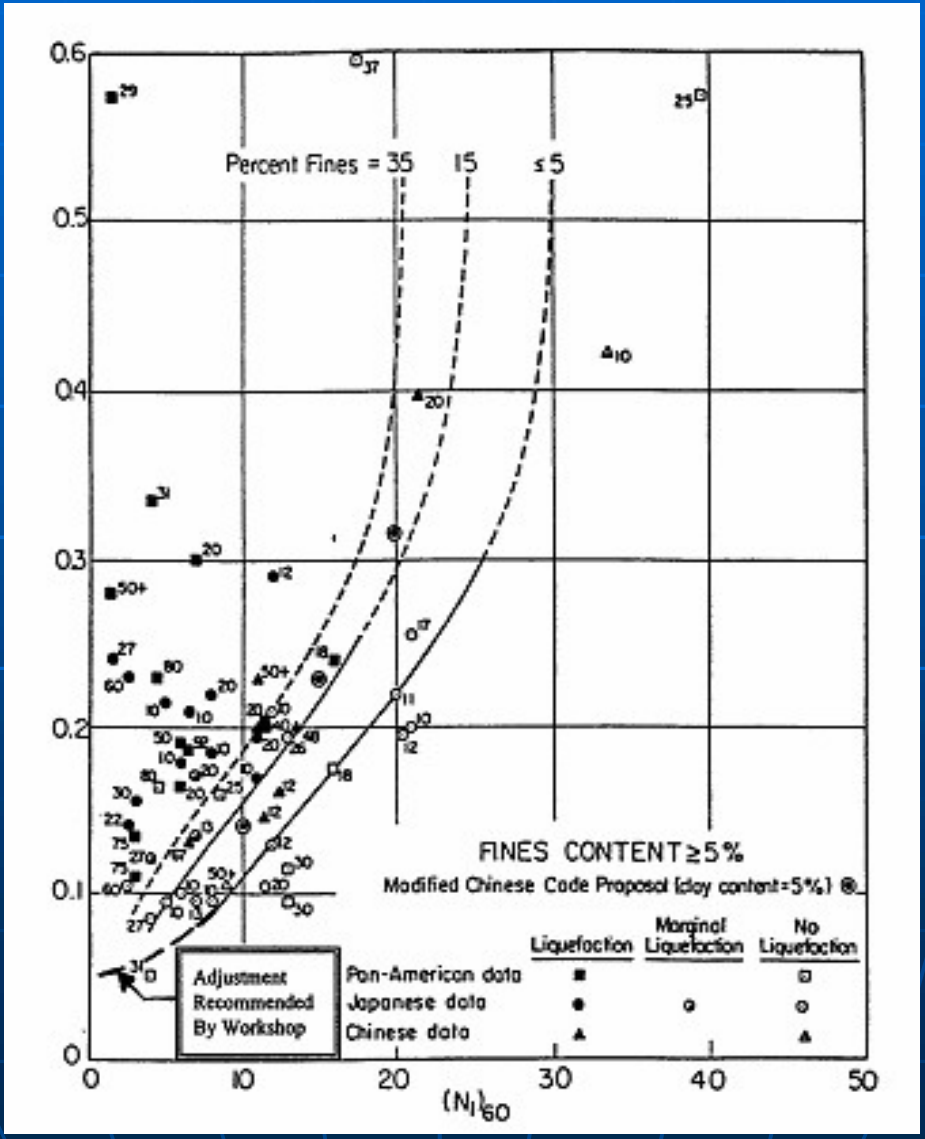
$$FS = CRR / CSR$$

CRR (Cyclic Resistance Ratio): Represents resistance of the soil against liquefaction based on SPT

CSR (Cyclic Stress Ratio): Represents liquefaction demand imposed by earthquake

$$(CSR = \tau_{cyc} / \sigma'_{v0})$$

Relationship between CRR and SPT-$(N1)_{60}$ (Seed et al., 1975)



$$CSR = \tau_{cyc} / \sigma'_{v0}$$

$$CSR = 0.65(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$$

$$r_d = \frac{(1 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5})}{(1 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.00121z^{2})}$$

$$CRR_{7.5} = \frac{1}{34 - N_{60}} + \frac{N_{60}}{135} + \frac{50}{(10 \cdot N_{60} + 45)^2} - \frac{1}{200}$$

Corrections to SPT (adapted from Youd et al., 2001)

Factor	Equipment variable	Term	*Correction*
Overburden pressure	-	C_N	$(P_a/\sigma'_{vo})^{0.5}$, $C_N \leq 1.7$
Hammer efficiency	Donut Hammer	E_m	0.4-0.6
Hammer efficiency	Safety Hammer	E_m	0.4-0.7
Hammer efficiency	Automatic trip Donut Hammer	E_m	0.5-0.85
Rod length	< 3m	n_1	0.75
Rod length	3-4 m	n_1	0.80
Rod length	4-6 m	n_1	0.85
Rod length	6-10 m	n_1	0.95
Rod length	10-30 m	n_1	1
Sampler Correction	Without liner	n_2	1
Sampler Correction	With liner: dense sand, clay	n_2	0.8
Sampler Correction	With liner: loose sand	n_2	0.9
Borehole diameter	65-115 mm	n_3	1
Borehole diameter	150 mm	n_3	1.05
Borehole diameter	200 mm	n_3	1.15

$$N_{60} = \frac{N_{field} \cdot C_N \cdot E_m \cdot n_1 \cdot n_2 \cdot n_3}{0.60}$$

Corrections to SPT (continued)

$$N_{60} = N_{field}\ C_N\ E_M\ N_1\ N_2\ N_3 / 0.60$$

where

N_{field} = measured standard penetration resistance

C_N = factor to normalize N_{field} to a common reference effective overburden stress of 1 Atmosphere (100 kPa)

E_M = hammer efficiency

n1 = correction factor for rod length

n2 = correction factor for samplers

n3 = correction factor for borehole diameter.

Corrections to SPT (continued)

- Another important factor is the energy delivered from the falling hammer to the SPT sampler.
- Hammer efficiency of 60% is accepted as a reference value for energy corrections.
- The energy ratio delivered to the sampler depends on the type of hammer, anvil, lifting mechanism, and the method of hammer release.
- Approximate hammer efficiencies for various types of hammers and anvils vary between 0.40 and 0.85.
- The corrections suggested for rod length, sampler and borehole diameter are also indicated in the table .

$$(N_{60})_{cs} = \alpha + \beta \cdot N_{60}$$

$(N_{60})_{cs}$: İnce malzeme oranı düzeltmesinden sonraki SPT darbe sayısı

α, β: ince malzeme oranı düzeltme katsayıları

$$\alpha = 0 \text{ for } FC \le 5\%$$
$$\alpha = \exp\left[1.76 - (190/FC^2)\right] \text{ for } 5\% < FC < 35\%$$
$$\alpha = 5 \text{ for } FC \ge 35\%$$

$$\beta = 1 \text{ for } FC \le 5\%$$
$$\beta = \left[0.99 + (FC^{1.5}/1000)\right] \text{ for } 5\% < FC < 35\%$$
$$\beta = 1.2 \text{ for } FC \ge 35\%$$

FC: İnce malzeme oranı

$(N_{60})_{cs}$<30 ise zemin sıvılaşma değerlendirilmesine tabi tutulur. **$(N_{60})_{cs}$≥30 olan zeminlerin sıvılaşmayacağı kabul edilir**.

$$MSF = 10^{2.24} / M_w^{2.56}$$

$$CRR_M = CRR_{7.5} \cdot MSF$$

MSF: Deprem magnitüdü düzeltme faktörü
M_w: Deprem magnitüdü

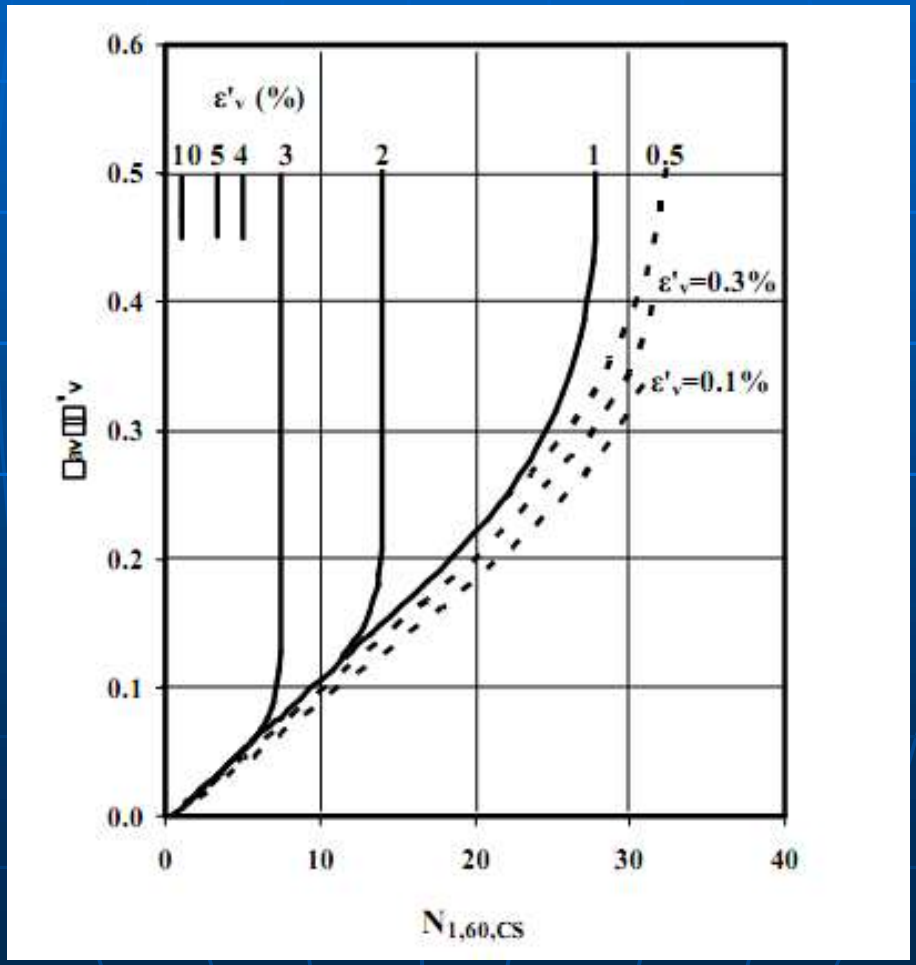
$$FS = CRR_M / CSR$$

Sıvılaşma Durumunda Zemin Parametrelerine uygulanacak Azaltma Faktörleri

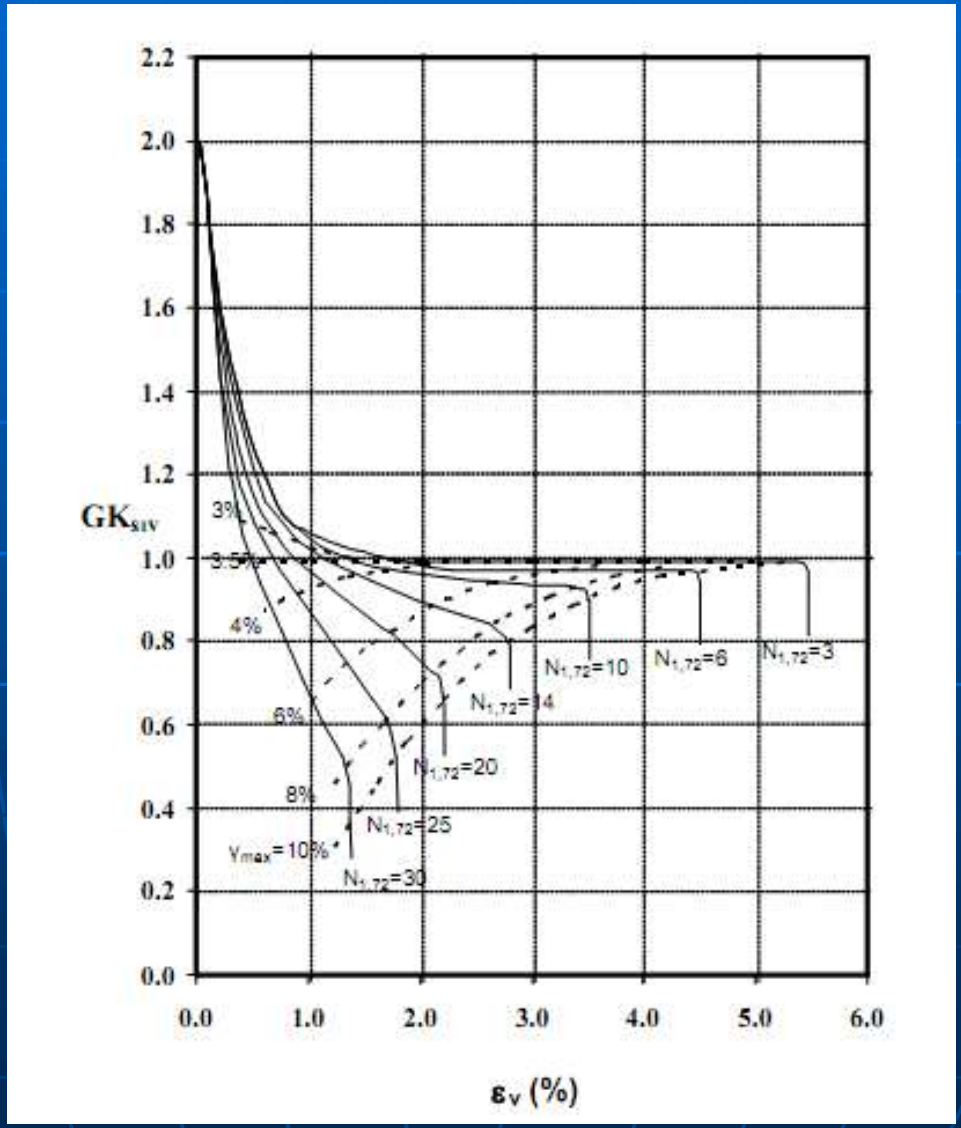
FS	Zemin yüzeyinden derinlik (m)	Azaltma Faktörü, D
FS≤0.6	0≤x≤10 10≤x≤20	0 1/3
0.6<FS≤ 0.8	0≤x≤10 10≤x≤20	1/3 2/3
0.8<FS≤ 1.0	0≤x≤10 10≤x≤20	2/3 1

Sıvılaşmadan sonra meydana gelecek oturmalar

Tokimatsu ve Seed (1984) tarafından önerilen hacimsel birim deformasyon ilişkileri



Ishihara ve Yoshimine (1992) tarafından önerilen hacimsel birim deformasyon ilişkileri



$$\sum s = \sum (\varepsilon_{v,i} \cdot t_i)$$

s: Sıvılaşmadan sonra beklenen oturma miktarı
t: İlgili tabaka kalınlığı