Development of the Earthquake Geotechnical Engineering (EGE) in ISSMGE

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After the original studies at the University of California led by H.B.Seed and I.M.Idriss, the study of EGE was introduced to ISSMFE in 1985 as one of the activities of a technical committee (TC) of ISSMFE. Since then, it has grown to an area of importance through the activities of TC-4 (1985 – 2009) and TC-203 (2009 -present).

Factor of Safety

$$F = \frac{\text{Capacity}}{\text{Demand}} = \frac{\text{Resistance of soils}}{\text{Earthquake induced force}}$$

・Capacity : Geotechnical Engineering

・Demand : Earthquake Engineering

⇨ (Earthquake Geotechnical Engineering) (Geotechnical Earthquake Engineering)

Fig.1. Geotechnical Engineering versus Earthquake Engineering

The development of the EGE can be considered roughly as a challenge to explore "capacity" versus "demand", as illustrated in Fig. 1. There has been a steadily increased demand coming mainly from the advances of earthquake engineering. Due to proliferation of the network of strong motion recorders, the magnitude of recorded acceleration in recent earthquakes has substantionally increased to a level as large as 1000 gal (1 g) as illustrated in Fig. 2.

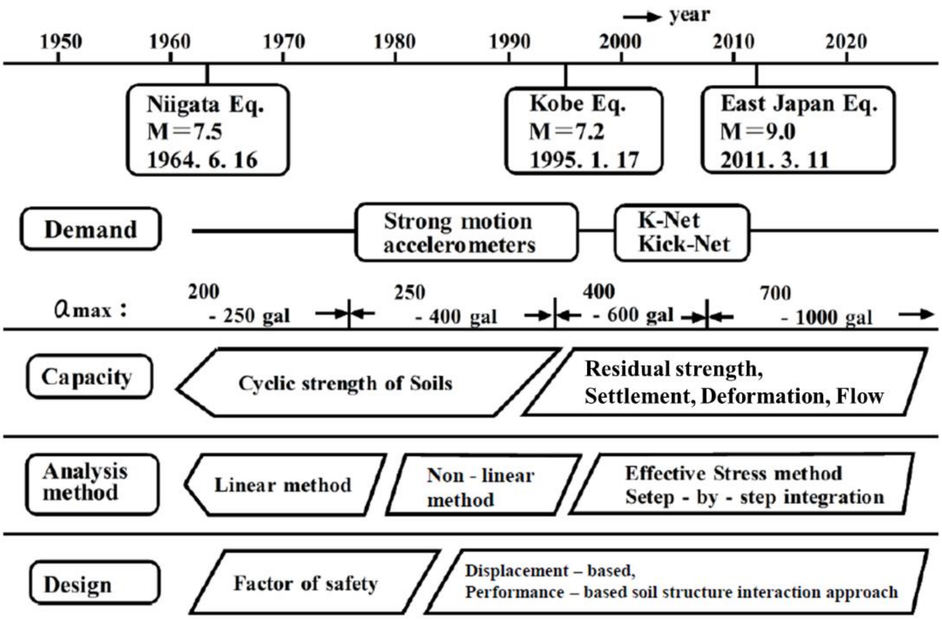

Fig.2. Changes in the Demand and Capacity

In response to the growing demand year after year, researchers and engineers in the geotechnical engineering discipline have been requested to take initiative and to cope with this increasing demand. This involved extensive endeavor on the capacity side as illustrated in Fig. 1. Detailed studies on the deformation characteristics, cyclic strength, residual strength and estimate of allowable displacements, etc. have been targets of studies in the geotechnical engineering. In the course of these efforts, the methodologies to evaluate the ground behavior has changed from the simple factor of safety approach, to linear and non-linear, and further to the analysis based on the effective stress, as illustrated in Fig. 2. In accordance to the general trends as above, the norm or protocol of design has changed from simple evaluation of factor of safety to displacement-based or performance-based approach as illustrated in Fig. 2.

The development of the earthquake geotechnical engineering has been enhanced through the quadrennial International Conference on Earthquake Engineering (ICEGE) as shown in Fig. 3. The names of the chair or Secretary General (SG) of the conferences and the chairperson in change of the operation of the TC for the 4 year interval period are shown in Fig. 3.

In the meanwhile, another conference series was inaugurated in 2009, that is, the International Conference on Performance-based Design in Earthquake Geotechnical Engineering (ICPBD) which is intended to focus more on the problems associated with practice in EGE. This conference series has been held in the mid-term year in the 4 year period of the ICEGE as shown in Fig. 3. The venues and names of the chairperson in charge of the conference are also shown in Fig. 3.

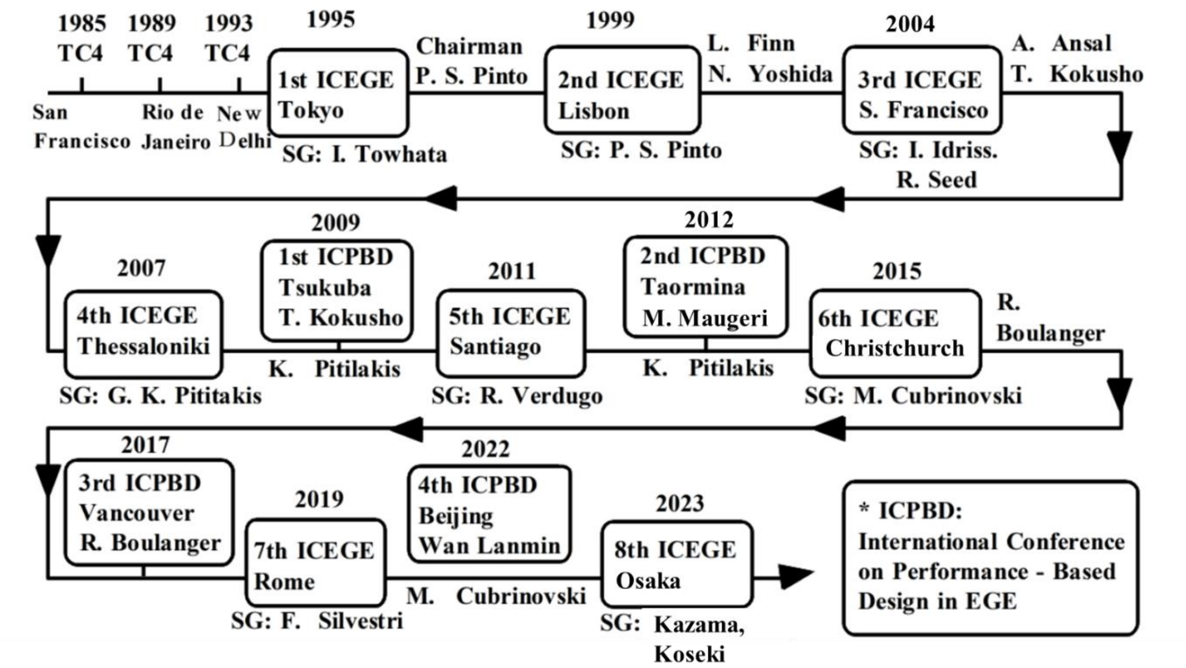

Fig.3. International conferences on earthquake geotechnical engineering

In addition, a lecture series named "Ishihara Lecture" was inaugurated in 2003. The list of the lecturers is shown below.

1st:
Finn W.D.L. 2003, at the 3rd ICEGE, San Francisco, USA
"An overview of the behavior of pile foundations in liquefiable and non-liquefiable soils during earthquake excitation"
2nd:
Idriss I.M. (paper with Boulanger, R.) 2007, at the 4th ICEGE, Thessaloniki, Greece
"SPT-and CPT-based relationships for the residual shear strength of liquefied soils"

3rd:
Dobry R. (paper with Abdoun, T.) 2011, at the 5th ICEGE, Santiago, Chile
"An investigation into why liquefaction charts work:
A necessary step toward integrating the states of the work：
A necessary step toward integrating the states of art and practice"
4th:
Gazetas G. 2013, at the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris, France
"Soil-foundation structure systems beyond conventional seismic failure thresholds"
5th:
Kokusho T. 2015, at the 6th ICEGE in Christchurch, New Zealand
"Liquefaction research by laboratory tests versus in-situ behavior"
6th:
Bray J.D. (paper with Macedo J.) 2017, at the 19th International Conference on Soil Mechanics and Geotechnical Engineering, Seoul, Korea
"Simplified procedure for estimating liquefaction-induced building settlement"
7th:
Towhata I. 2019, at the 7th ICEGE, Rome, Italy
"Summarizing Geotechnical activities after the 2011 Tohoku Earthquake of Japan"
8th:
Cubrinovski M. (paper with Ntritsos N.) 2022, at the 4th ICPBD, Beijing, China
"Holistic evaluation of liquefaction problems"

Fundamental laws for deformation of granular soils

While the basic laws of deformation had been established for cohesive soils and incorporated into the framework such as the Cam-clay model and critical-state soil mechanics, studies in a similar vein were lacking and left behind for cohesionless soils such as silt, sand, and gravel. However, since the issues of sand liquefaction were recognized as an important problem after the occurrence of the Niigata earthquake in 1964, needs were instigated for studying the basic features of deformation mechanism for the cohesionless granular soils particularly under dynamic and cyclic loading conditions. The trend in the main stream of development in soil dynamics is briefly illustrated in Fig. 4, as contrasted to the advances in soil mechanics which are mainly associated with the static loading condition. It is to be noted that, while the soil mechanics has been developed by considering the soil mass as a continuum, the same thought has also been adopted as the basis for developing theorems of deformation for granular soils. The constitutive laws of deformation for the granular soils have been known to be composed of four basic laws, that is, 1) Friction law, 2) Dilatancy law, 3) Flow rule, and 4) Harding rule.

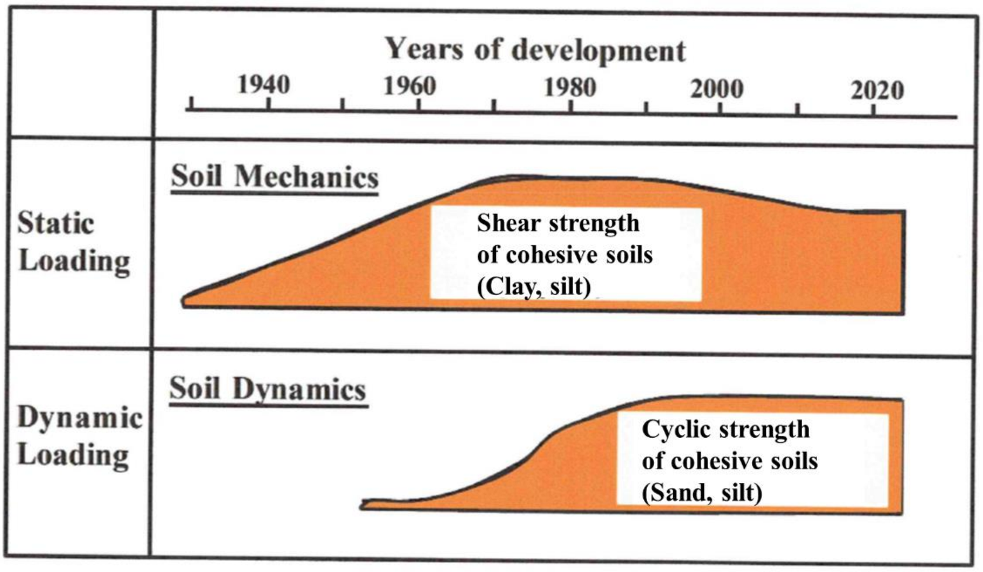

Fig. 4. Development of Soil dynamics in Comparison to that of Soil mechanics

1) Friction Law

In the early period of studies in late 1960s at the University of Tokyo, a set of data was obtained on sand using the triaxial test apparatus[1)]. In the classical theory of plasticity developed for the deformation of steel or concrete, the yielding is assumed to take place merely with increase in shear stress. In contrast, the irrecoverable permanent deformation in the granular soils was shown to occur, only with increase in the stress ratio, as illustrated in Fig.5, leading eventually to the failure criterion known as Coulomb law of friction. The results of the tests are considered as the basis of seminal importance to substantiate validity of the “friction rule” for granular soils.

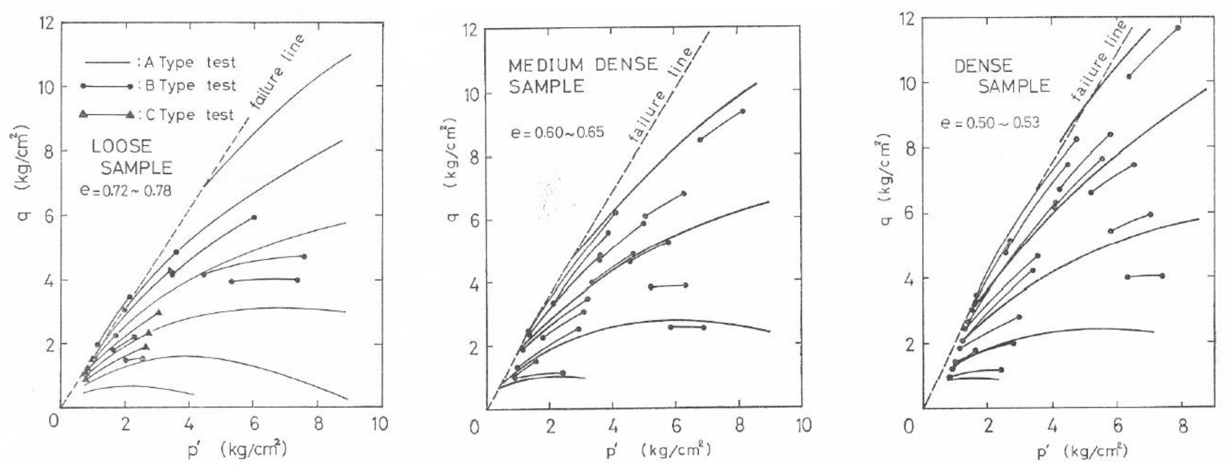

Fig. 5. Yield loci for granular soils[1)]

2) Dilatancy Law

As well-known, significant volume change can occur easily in the granular soils not only by compressive stress but more predominantly by the application of shear stress. The dilatancy as above has been investigated thoroughly and formulated in various fashions by several investigators in the early period of development in soil mechanics. The most widely used formula is the relation, as follows, which was derived from the energy balance during the stress application.

$$\frac{\Delta\varepsilon_X + \Delta\varepsilon_Z}{\sqrt{\Delta\gamma^2_{XZ} + \left(\frac{\Delta\varepsilon_Z - \Delta\varepsilon_X}{2}\right)^2}} = M - \frac{\sigma_Z - \sigma_X}{\sigma_Z + \sigma_X} \quad \cdots\cdots \quad (1)$$

where $\Delta\varepsilon_x, \Delta\varepsilon_z$ and $\Delta\gamma_{xz}$ are increments of plastic strain component, and $\Delta\sigma_x$ and $\Delta\sigma_z$ are stress increment-components, as illustrated in Fig. 6. It is to be noted that when the current shear stress ratio is smaller than M, the volume change is positive, that is, volume decrease takes place. In contrast, if the current shear stress ratio is larger than M, the increase in volume takes place.

3) Non-coincidence of the principal axes of stresses and plastic strain increments.

In executing the stress and strain analysis in the two-dimensional plane strain conditions, what is tacitly assumed in the classical theory of metal plasticity is what is called “flow rule” which is formulated as below.

$$\frac{\tau_{zx}}{(\sigma_Z - \sigma_X)/2} = \frac{\Delta\gamma_{zx}}{(\Delta\varepsilon_Z - \Delta\varepsilon_X)/2} \quad \cdots\cdots\cdots \quad (2)$$

As illustrated in Fig. 6, Eq. (2) implies that the vector formed by the two components of plastic strain increments should be directed in the same direction as the vector formed by the two components of current state of stresses. In this context, the flow rule is alternatively called “coincidence of principal axes”.

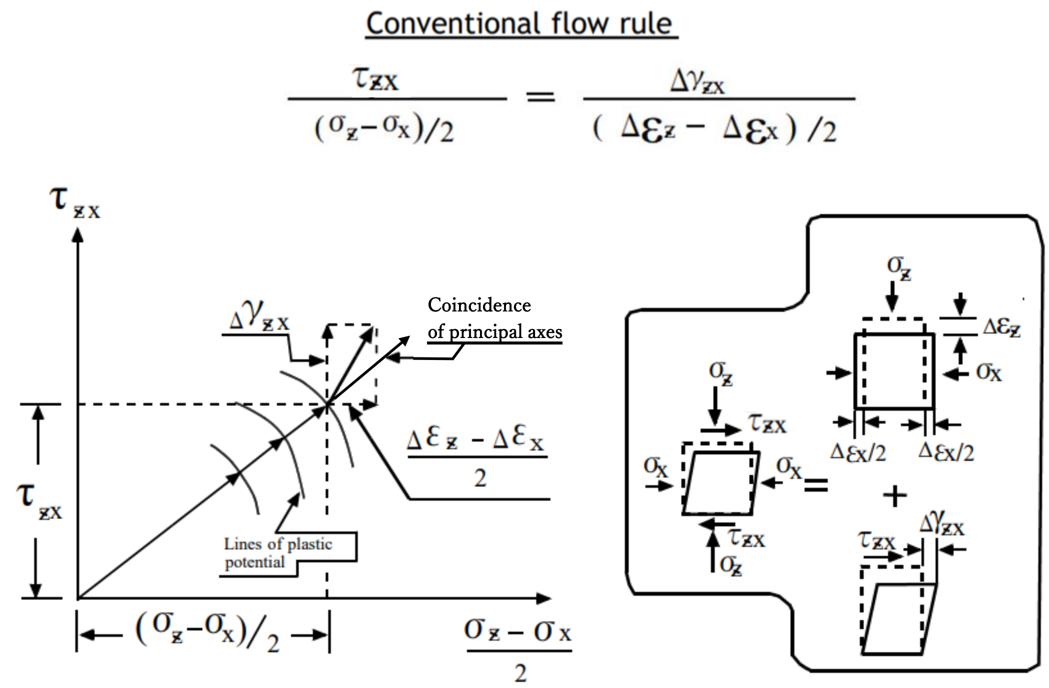

Fig. 6. Non-coincidence of principal axes of shear stresses and plastic strain increments.

However, as a result of comprehensive laboratory tests, it has become clear that the rule as above does not hold valid for the granular materials such as sand and gravel which are deposited under gravity. The laboratory tests using the triaxial torsion tests have shown the results as typically demonstrated in Fig. 7. Pending detailed description here, a new method for constructing the flow rule was proposed by the researchers at University of Tokyo.[2] It is to be noted that the effects of non-coaxiality could exert some influence particularly when dealing with the sand performance subjected to cyclic stresses involving continuous rotation of principal stress axis such as those under wave-induced or traffic induced loads.[3]

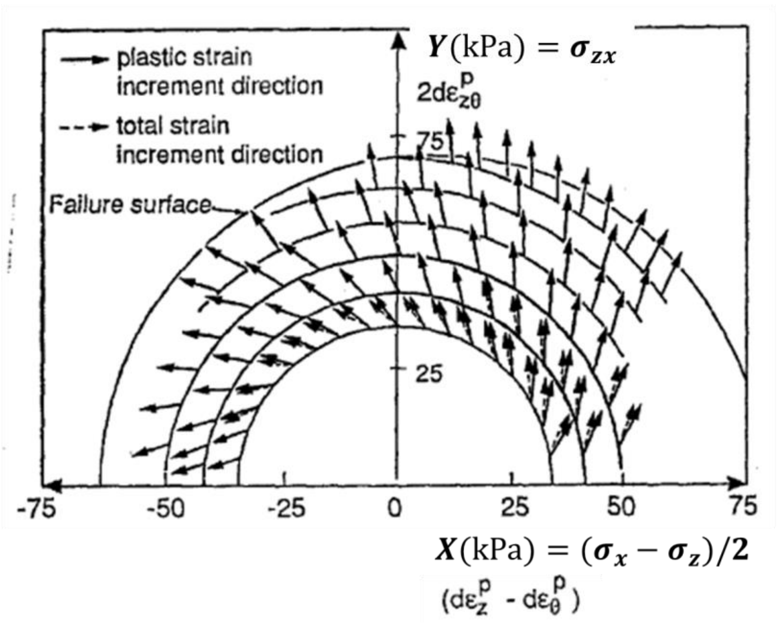

Fig. 7. Plastic strain development at various stress states.[2]

4) Plastic stress-strain relation

In addition, to conduct the effective stress analysis in 2D condition, what is called "hardening rule" needs to be specified. Various relations have been used, but the hyperbolic type relation is used most commonly. Details of the hardening function are omitted here.

References

(1) Tatsuoka, F. and Ishihara, K. (1974), "Yielding of Sand in Triaxial Compression", Soils and Foundations, Vol.14, No.2, pp.63-76

(2) Gutierrez, M., Ishihara, K. and Towhata, I. (1991)", Flow Theory for Sand during Rotaion of Principal Stress Direction", Soils and Foundations, Vol. 31, No. 4, pp. 121 – 132.

(3) Ishihara, K. (1983), "Soil Response in Cyclic Loading Induced by Earthquakes, Traffic and Waves", Proc. 7th Asion Regional Conference on Soil Mechanics and Foundation Engineering, Haifa, Israel, Vol. 2, pp.42 – 66.