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PLASTICITY INDEX EFFECTS ON MECHANICAL BIFURCATION: SOILS AND SOFT-SEDIMENT DEFORMATION

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ABSTRACT: Yielded specimens have various deformation patterns depending on loading stages and stress ratios in spite of the same geomaterials (e.g. soils and soft-sediment). Failure patterns of materials bifurcationally change to diamond, bulge and a pair of oblique shear patterns. Symmetry of deformation patterns (e.g. shear band patterns) has been illuminated by bifurcation analysis of governing equation based on Cam-clay model. Plasticity index tested by Casagrande liquid limit device and plastic limit instruments is known to describe mechanical characteristics (e.g. compressibility) of soils. Relationship between plasticity index and mechanical bifurcation controls the evolution of deformation patterns. From the view point of concept of plastic potential, we show that plasticity index theoretically determines deformation patterns of soils on the basis of the bifurcation analysis. Furthermore, because deformation facies of rocks are described by mean ductility of rocks similar to the concept of plasticity index, we point out that mean ductility controls mechanical bifurcation of the geologic materials such as soft-sediments.

Keywords: Soil, Critical state soil theory, Plasticity index, Cam-clay model, Bifurcation

1. INTRODUCTION

1.1 Theoretical and experimental preface

When deformation phenomena of geomaterials such as soils, soft-sediments, rocks, and even tectonic plates are described, we generally use Mohr's circle or simulations based on Cam-clay model. However, it is difficult to explain soil parameters controlling deformation phenomena from the view point of geology and soil mechanics. The reason for the difficulty is that physical properties controlling on deformations of soils and rocks cannot be determined uniquely. For example, Paterson [1] used cylindrical marbles under various confining pressures up to 98 MPa by triaxial compression test. Significantly, the developed shear zones display three different patterns in spite of the same starting materials. With respect to these fracture developments, Shibi and Kamei [2] and Ikeda *et al*. [3] pointed out that the surface fractures of cylindrical specimens of geomaterials bifurcationally changes to diamond pattern, bulge pattern and a pair of oblique shear patterns. Plasticity index I_P is an important parameter in Shibi and Kamei [2]'s analysis, so we theoretically introduce that the index can be theoretically related to mechanical formulae and Cam-clay model based on elastoplasticity.

The plasticity index has been a conventional and simple experimental parameter to evaluate mechanical properties of soils. Voight [4] and Kanji [5] mentioned direct correlations between internal friction angle φ and plasticity index I_P. Slope stability depends on I_P related to compressibility by water content. On the other hand, the relationship between I_P and mechanical parameters such as mean and deviatoric stresses, and the theoretical relationships between soil strength φ and I_P have been derived [4]–[7]. Interestingly, Uemura [8] proposed that classification of folding can be expressed by mean ductility of rocks similar to the concept of I_P.

Firstly, we introduce the concept of plastic potential, critical state soil theory, and plasticity index. Secondly, the relationship between the mechanical bifurcation and the plasticity index are briefly shown. Thirdly, we show the result of soil tests for geomaterials. Finally, we discuss the mechanical bifurcation determined by the concept of I_P and implication of deformation of geomaterials. This paper is an extended paper of the Proceeding of the Second International Conference on Science, Engineering and Environment (SEE-Osaka 2016) [9] with some modifications.

1.2 Concept of plastic potential

Cam-clay model can be derived from flow rule and some constitutive laws [2]. Flow rule used by plastic potential f is given by [10];

$$d\varepsilon^p = h\frac{\partial f}{\partial \sigma'_{ij}}, \quad h > 0, \tag{1}$$

where $d\varepsilon^p$ is incremental tensor of plastic strain rate, σ'_{ij} is effective stress tensor, and h is plastic multiplier. Increment tensor of plastic strain is partial differential of the potential due to effective stress in q–p'–v space, where q is the deviatoric stress, v is the specific volume and p' is the mean effective stress. In q–p'–v space, the condition of soils and rocks under triaxial compression is expressed by critical state line. When stress state of soils reaches to the critical state, the soils yields and behaves fluidity. Regarding plastic potential on q–p' plane, the critical state line shows plastic expansion on the upper side of the line and shows plastic compression on the lower side of the line. Therefore, the critical state line can be understood as a boundary line of conditions. Then, the slope of critical state line by M is defined by;

$$q = M p'. \tag{2}$$

The work increment at shear deformation is assumed to equal the energy dissipation at the critical state [11], [12]. Furthemore, Eq. (2) is displayed as follows,

$$p' d\varepsilon_v + q d\varepsilon_d = M_c p' d\varepsilon_d, \tag{3}$$

where M_c is constant, ε_v is volumetric strain and ε_d is deviatoric strain. From Eq. (3), we can obtain,

$$\frac{q}{p'} = M_c - \left(\frac{d\varepsilon_v}{d\varepsilon_d}\right). \tag{4}$$

This is the relationship between stress ratio and stain increment ratio in Cam-clay model [11], [12], which is similar to Eq. (2). Rowe [13] microscopically studied Eq. (4) from the view point of deformation of granular materials, and obtained a similar relation as,

$$\frac{\sigma_1}{\sigma_3} = K\left(\frac{-d\varepsilon_3}{d\varepsilon_1}\right) = -\frac{2\dot{\varepsilon}_3}{\dot{\varepsilon}_1}\tan^2\left(\frac{\pi}{4}+\frac{\varphi}{2}\right), \tag{5}$$

where σ_1 is major principal stress, σ_3 is minor principal stress, ε_1 is axial strain, and ε_3 is lateral strain. This is an expression for the relationship between principal stress ratio (= σ_1/σ_3) and principal strain incremental ratio during the shear of granular materials (= $-d\varepsilon_3/d\varepsilon_1$). This is usually called stress-dilatancy relation. Further, K is internal energy ratio of soil. Besides, we can regard Eq. (2) or Eq. (4) as Eq. (5). Then, normal vector of plastic potential plane is equivalent to the yield function of $d\varepsilon^p$ on q–p' plane. Therefore, we obtain

$$\frac{d\varepsilon_v^p}{d\varepsilon_d^p} = -\frac{dq}{dp'}. \tag{6}$$

From Eqs. (4) and (6), we can get

$$\frac{q}{p'} - \frac{dq}{dp'} = M_c. \tag{7}$$

Now, setting from the case ($p' = p_y$, $q = 0$; p_y is a hardening parameter) and integrating Eq. (7), we get

$$f = \frac{q}{M_c p'} + \ln\left(\frac{p'}{p'_0}\right) - \ln\left(\frac{p_y}{p'_0}\right), \tag{8}$$

where f is the yield function of Cam-clay model. As shown in the present study, Cam-clay model can be derived from flow rule and some constitutive laws. By using p_y, the increment of plastic void ratio e^p is given by,

$$de^p = -(\lambda - \kappa)\frac{dp_y}{p_y}, \tag{9}$$

where κ is swelling index, and λ is compression index at the critical state line [12].

On the other hand, the relationship between the increment of plastic volumetric strain $d\varepsilon_v^p$ and plastic volume ratio dv^p is given by,

$$d\varepsilon_v^p = -\frac{dv^p}{v} = -\frac{de^p}{1+e}, \tag{10}$$

where specific volume v is expressed by void ratio e ($v = 1 + e$). Eq. (10) can derive the relationship between precursory phenomena for earthquake and consolidation based on Terzaghi's consolidation equation as equivalent to diffusion-like equation of the strain [14].

Projecting Eq. (2) onto a v–p' plane based on the critical state soil theory [11], [12], we can obtain

$$v = \Gamma - \lambda \ln p', \tag{11}$$

where Γ is constant. Moreover, the normal consolidation line is given by

$$v = N - \lambda \ln p', \tag{12}$$

where N is constant. This line runs parallel to Eq. (11). In general, consolidation phenomenon is closely linked by the change of e. Now, we show theoretically that Eqs. (11) and (12) are related to e determined by Eq. (10). Terzaghi's consolidation theory explains Eq. (12) that change void ratio depending on time.

1.3 Experimental definition of plasticity index

For more consideration about consolidation phenomenon, we need to confirm that I_P is essentially connected to v. These directly link to the compressibility of soils. Compression index C_c representing the compressibility of soil can be obtained by

$$e = e_0 - C_c \log p', \tag{13}$$

where e_0 is initial void ratio. C_c is expressed by

$$C_c = \frac{I_P}{74}, \tag{14}$$

where I_P is defined by non-dimensional units as follows;

$$I_P = w_L - w_P, \tag{15}$$

where w_L means liquid limit [%]: the water content at which the soil ceases to be liquid, and w_P means plastic limit [%]: the water content at which the soil ceases to be plastic [15], [16]. Plasticity index I_P is an empirical parameter to characterize the range of water contents where the soils exhibit plastic property, and is known to describe mechanical characteristics (e.g. compressibility) of soils.

On the other hand, I_P determines mechanical bifurcation controlling the evolution of deformation patterns [2]. Based on Eq. (14), the more consistency of soils increases, the more compressibility increases substantially than several tens to hundred times. Moreover, I_P links to the parameters of Cam-clay model widely used for simulation of consolidation and shearing of clay.

1.4 Mechanical bifurcation in Cam-clay model: I_P and mechanical parameters

Bifurcation analysis can be applied to failure patterns of geomaterials [2], [3]. Under the condition of mechanical parameters, deformation patterns develop characteristically.

Shibi *et al*. [17] analyzed that the initial imperfection was introduced into samples based on a coaxial Cam-clay model. Then, the function of coaxial Cam-clay model f_c is given by

$$f_c = \frac{\lambda - \kappa}{1+e} \ln \frac{p'}{p'_0} + \tilde{D}\frac{q}{p'} - \varepsilon_v^p, \tag{16}$$

where $\tilde{D}$ is coefficient of dilatancy. The ratio of q to p' is called stress ratio η (= q/p') expressed in dimensionless unit. When deformation patterns bifurcate, the stress ratio is defined as bifurcation load η_y [2]. That is to say, Eq. (16) describes that deformation of rocks and soils is controlled in q–p'–v space. Based on Cam-clay model [2], [18], bifurcation behavior of the soil with the lower I_P occurs at larger η and smaller ε_1 than those with higher I_P. I_P related to C_c can be theoretically expressed by λ (Eq. 11). In addition, some soil parameters (λ, κ, M) in Cam-clay model (Eq. 16) can be linked to I_P. Significantly, I_P closely affects critical state theory from specific volume v (= 1 + e). Moreover, various soil parameters can be determined simply by I_P because of linear correlations among them as shown in Table 1.

Table 1 Soil parameters and plasticity index

$\lambda = 0.02 + 0.0045 I_P$
$\kappa = 0.00084\,(I_P - 4.6)$
$N = 1.517 + 0.019 I_P$
$M = 1.65$
$\tilde{D} = 0.00082 I_P + 0.0159$

Nakase *et al*., [19] and Kamei [20] tested the reconstituted twelve soils which range of I_P from 10 to 55.

1.5 Bifurcation analysis based on Cam-clay model

By means of the bifurcation analysis based on a coaxial Cam-clay model, deformation patterns of specimen are revealed by simulation [17], [21]. Shibi and Kamei [2] derived a governing equation from flow rule including plastic potential and constitutive law.

The velocity v_i, in the coordinate system on (x_i) = (x_1, x_2) ≡ (x, y) with, i = 1, 2 is defined by the stream function ψ as

$$v_1 = \psi_{,2},\; v_2 = -\psi_{,1}. \tag{17}$$

By using the stream function in Eq. (17), a governing equation of ψ can be given by

$$a\psi_{,1111} + b\psi_{,1122} + c\psi_{,2222} = 0, \tag{18}$$

where the constitutive parameters of a, b and c are determined by virtue of I_P through various parameters shown in Table 1 [2]. Moreover, the results of Shibi and Kamei [2] indicates the influence of soil parameters on bifurcation behavior of normally consolidated cohesive soils under plane strain undrained compression loadings, and they applied to the prediction of slip surface. Then ψ can be expressed by

$$\psi = V(x_1)\cos(k_m x_2),\; k_m = \frac{m\pi}{2H}, \tag{19}$$

where $V(x_1)$ is the general solution at x_1 coordinate, k_m is the function of deformation mode m, and H is height of specimen. By Eq. (19), Eq. (18) can be written by

$$(aD^4 + 2bk_m{}^2 D^2 + ck_m{}^4)V(x_1) = 0, \tag{20}$$

where D is differential [2]. Then, the governing equation can expressed by

$$a\rho^4 + 2b\rho^2 + c = 0, \tag{21}$$

where ρ is bifurcation solution [20].

2. SAMPLES AND METHODS

In this section, the physical properties of geologic materials are measured because there is not enough data of the materials for soil tests. Firstly, Futaba-fault gouge (Jisahara, Minamisouma city, Fukushima prefecture, Japan [22]) is used to investigate mechanical properties of active fault gouge. Moreover, we also measure the physical property of volcanic ashes (Mt. Aso, Kumamoto prefecture, Kyusyu, Japan) which may cause disastrous lahars with fluidity by rain. The ashes are picked at Mt. Aso (Kumamoto prefecture, Kyusyu, Japan). On the other hand, the landslide surface is potentially formed by the trigger of porcelain clay consisting of fine-granited soil consists of fine grains and is deposited in the Chubu region (Central Japan). We used two porcelain clays (Gaerome and Kibushi clay). The porcelain clay was lake deposits. This study focused on the Lake Tokai (Mogusa Tsuchi clay, Mino Kibushi clay, Seto Gaerome clay, and Seto Honzan Kibushi clay), existed at Gifu and Aichi prefecture (Chubu, Japan), and the Ancient Lake Biwa (Ao Gaerome clay and Shigaraki Mizuchi clay), located on the Mie prefecture, (Chubu, Japan) [23]–[25]. Two types of clay minerals (halloysite and montmorillonite) are also investigated in this study.

Secondly, we measured physical parameters of soils through experiments, and especially noticed the consistency signifying the physical state of soils. The soil tests are followed by Japanese Industrial Standards (JIS). Samples are well-mixed for adjusting the grain size and uniformity property. According to the JIS A 1202, the density of particles of soil ρ_s can be determined by the particles density test with a pycnometer. For estimating plastic index I_P, two tests are used to determine the liquid limit w_L and plastic limit w_P according to JIS A 1205. In the liquid limit test, samples with various water contents are placed in a brass cup. Then a number of blows is counted until the two separated parts of the soil sample on the cup come into contact. From the relationship between numbers of brows and water contents for various samples, the water contents at 25 blows are determined to be a liquid limit. For the plastic limit, the samples are rolled on a glass plate using the palm of hand until crack formation occurs at 3 mm diameter. Then the water contents of the samples at the crack formation are calculated to be the plastic limit of the soil. The liquid limit and plastic limit of sandy and silty soil cannot be determined, and hence the soil consistency is called non-plastic; NP [26], [27]. The w_P and w_L express the physical states of soils. Moreover, we conducted the ignition loss L_i test for measuring the content of organic matter of the soils according to JIS A 1226.

3. RESULTS

The results of soil tests are as listed in Table 2. In comparison with density and consistency, Mino Kibushi clay has similar to the parameters to those of halloysite except their ignition losses L_i. The difference in density between Mino Kibushi clay and halloysite are only 0.02 g/cm^3, also the difference of consistency is under 2%. Then, the parameter of L_i

Table 2 Results of soil tests for geologic materials

	Density ρ_s (g/cm^3)	Liquid limit w_L (%)	Plastic limit w_P (%)	Plasticity index I_P	Estimated φ (°)	Ignition loss L_i (%)
Halloysite	2.63	44.93	26.23	18.71	12.5	12.30
Montmorillonite	2.81	1042.13	80.94	961.19	2.1	4.99
Mt. Aso volcanic ash (Mt. Naka)	2.75	NP	NP	NP	–	-0.27
Mt. Aso volcanic ash (Hino Pass.)	2.74	NP	NP	NP	–	-0.30
Futaba-fault gouge	2.72	40.64	7.80	32.84	9.8	6.32
Mogusa Tsuchi clay	2.65	42.44	22.50	19.96	12.2	3.80
Mino Kibushi clay	2.61	46.51	28.60	17.91	12.8	9.94
Seto Gaerome clay	2.62	52.43	22.10	30.31	10.1	7.69
Seto Honzan Kibushi clay	2.55	59.90	28.10	31.80	9.9	12.62
Ao Gaerome clay	2.64	40.10	20.40	19.70	12.3	4.13
Shigaraki Mizuchi clay	2.66	37.75	21.10	16.67	13.2	4.47

of Seto Honzan Kibushi clay is similar to the parameter of halloysite except its ρ_s and consistency. Moreover, montmorillonite has high I_P because of structural feature (layer structure of the mineral). From this result, the permeability of montmorillonite is expected to be extremely low. Based on these findings, the consistency of various geomaterials differs from sedimentary/formation environments, constituent mineral types, and weathering. The reason for the non-plasticity (NP) of Aso volcanic ashes is that these aches are not weathered well, and behave as dry sand when it is dry. Therefore, it is difficult to measure its liquid/plastic limit. On the other hand, the ignition loss for volcanic ashes has negative value. The ignition loss represents the content of organic matter in the soil, consequently, two ashes do not have organic matter. It is thought that the mass is increased by oxidized metals in the soil under high temperature during the test (temperature of kiln; 750 ± 50 degrees Celsius).

4. DISCUSSIONS

4.1 Bifurcation determined by the concept of I_P

Based on above discussion, we consider physical meaning of I_P theoretically. When the Mohr-Coulomb's failure criterion is reviewed, void ratio e of the soil is not included. Moreover, the lateral pressure corresponding to p' in triaxial compression test relates to the internal friction angle φ obtained by the test. The φ is given by Eq. (5). Strains ε_1 and ε_3 can be differentiated by time. Hence, Eq. (5) shows that energy ratio can be a new factor for frictional instability such as deformation of fault gouge [28]. In addition, the empirical relationship between I_P and φ is approximately given by

$$\varphi = \alpha I_P^{-\beta}, \tag{22}$$

where α and β are specific values on soil types (e.g. α = 46.6 and β = 0.446; [5]).

Based on Eq. (5), we can obtain

$$\varphi = -\frac{\pi}{2} + 2\tan^{-1}\left(\sqrt{\frac{\sigma_1\dot{\varepsilon}_1}{2\sigma_3\dot{\varepsilon}_3}}\right). \tag{23}$$

From Eqs. (15), (22) and (23), we can get

$$I_P = \left(\frac{\alpha}{2\tan^{-1}(\sqrt{K}) - \pi/2}\right)^{\frac{1}{\beta}}. \tag{24}$$

In this way, I_P is linked to Eq. (24) through Rowe's energy ration of granular material. On the other hand, deformation mode can be analyzed. Significantly, parameters a, b and c in Eqs. (18), (20) and (21) relate to I_P. Eventually, deformation mode analysis, by the governing equation derived from flow rule based on plastic potential, is determined by I_P. In this paper, we detected the procedure of dealing with the view point of I_P in regard to deformation patterns. From the above, we evaluate that I_P theoretically determines the bifurcationally deformed patterns of soils.

The φ can be approximately estimated by I_P based on Eq. (22). From the viewpoint of friction angle, the landslide is predicted to happen in condition that friction angle is around 10° or less [29]. Interestingly, the φ of each porcelain clay and Futaba-fault gouges are also low. Kuwahara and Hirama [30] indicated fault gouges behave as a water barrier sheet partially to surrounding jointed bedrocks because the permeability of faults was low (also mentioned by [31]–[33]). Hence, application of liquid/plastic limit tests to soft-sediment is able to assess the basic mechanical data of materials such as φ and/or cohesion evaluating a physically weak zone in ground and geology conditions.

On the other hand, based on Yamaguchi *et al*. [34], mathematical conversion [35] related L_i to C_c by Eq. (13). Therefore, by Eq. (14), I_P can be linked to L_i. So, in the future, we need to study why organic matter is a parameter of Cam-clay model.

4.2 Deformation implication materials of soils and rocks

Finally, we discuss deformation patterns of geomaterials as soils and rocks through the concept of I_P. We can discuss the relationship between the concept of I_P and "Deformation facies diagram". The deformation facies proposed by [8] are represented by means of their series and grades on the coordinate axes. Uemura [8] said, "*The former allows the discrimination of the deformation series in a definite environment, and the latter is expressed by the deformation grade for a definite material. Ductility contrast and mean ductility are available as indices to qualify such series and grades respectively [36]. Taking into account that most rocks are not homogeneous in lithology but have some internal structures or they consist of multilayered strata, the ductility as a whole should be expressed by the average of constituent portions and layers. This is mean ductility. On the other hand, internal movement of rocks, which results in the deformation fabric, is expected to be largely dependent on the relative difference of ductility between adjacent constituents, namely it is the second index ductility contrast*". By the Uemura [8]'s proposal, deformation facies diagram shows that stratum including rocks is that fluidly deformed when its character change more ductile. The Uemura [8]'s concept versatilely represents the various deformation patterns of various rocks. However, the indices of mean ductility and ductility contrast are qualitative ones and hence difficult to argue the development of deformation patterns quantitatively. In deformation facies diagram, the vertical axis defines mean ductility and illustrated interestingly deformation patterns of specimens. On this vertical axis, mean ductility controls the deformation patterns: a pair of oblique shear pattern-diamond pattern-bulge pattern from low to high mean ductility. In other words, the plasticity index is important to determine patterns of folds and tectonics besides mechanical parameters (e.g. stress and strain). It should be noted that I_P is an index characterizing the state of soils, especially clay. However, we point out the possibility on that we can mesoscopically discuss the evolution of deformation patterns of the geologic materials such as soft-sediment deformation in virtue of I_P. For example, geological observation of soft sediments in subduction zones [37] reveals that the mode of sediment deformation is complex, and incompletely understood as an interaction of porosity, pore pressure, and state of accreting sediments. We show that the porosity of soft sediments corresponds to plasticity index of soil, and found the void ratio of soils can be related to porosity of rocks. Moreover, the deformation of strata is closely controlled by competent layer (sandstone) and incompetent layer (mudstone). This is, namely, ductility contrast can be explained by composition of the ratio of mud and sand. In conclusion, the geological deformation phenomena (including folds, tectonics and landslides) are not constantly controlled by physical parameters such as stress or strain. The plasticity index I_P determines deformation patterns.

5. CONCLUSION

Firstly, we showed that I_P theoretically determines deformation patterns of soils by Cam-clay model. Secondly, we prove that the index closely affects the bifurcation formulas by [2] and mention that plasticity index can reflect a mechanical bifurcation of rocks (e.g. variety of folds or soft-sediments). By theoretically connecting I_P, we might discuss landslides with lower φ from the view point of the physical mean of I_P.

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