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Quantitative Assessment on the Variation of Compressibility of Wenzhou Marine Clay During Destructuration

Xia Bian*, Jian-Wen Ding**, Jian Shi***, and Sen Qian****

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Abstract

A series of oedometer test were performed on both undisturbed and remolded specimens of Wenzhou marine clay to quantitatively investigate the degradation of soil structure during compression. The laboratory tests show that the swell index of both natural and remolded Wenzhou marine clay increase with the increase in consolidation stress. Hence, the normalizing parameter 'swell sensitivity', termed as the ratio of the remolded to the natural swelling index, can only be regarded as a qualitative parameter. On the other hand, the normalized stress sensitivity has a good parabolic relationship with the consolidation stress during destructuration. This result indicates that the stress sensitivity can be used as a quantitative interpretation of the degradation of soil structure for natural Wenzhou marine clay. Comparison between stress sensitivity and additional void ratio during destructuration shows that the variation of soil structure during destructuration represented by stress sensitivity and additional void ratio are essentially the same. In addition, the parameter used in formulating the variation of stress sensitivity with the stress level is correlated with natural void ratio and void ratio at liquid limit.

Keywords: *undisturbed clay, soil structure, compressibility, stress sensitivity, swell index*

1. Introduction

Soft marine clays are widely distributed along the coastal region of East Asia, such as Kwangyang and Busan marine clay in Korea (Yoon and Kim, 2006; Chung *et al.*, 2012), Kobe marine clay in Japan (Shibuya *et al*., 2008), Shanghai and Wenzhou marine clay in China (Sun *et al.*, 2014; Yin *et al*., 2015). Due to the rapid development of economy, many infrastructures need to be constructed over this region (Shibuya *et al.*, 2008; Kishore *et al*., 2009; Kulkarni *et al*., 2010; Wang *et al*., 2013). Hence, the mechanical behavior of natural marine clay is of great interest. It has been well documented that natural marine clays are generally subjected to the effects of soil structure during depositional and post-depositional processes (Leroueil *et al*., 1979; Leroueil and Vaughan, 1990; Cotecchia and Chandler, 1997, 2000; Chandler, 2000). The term 'soil structure' is used to illustrate the effects of fabric and bonding on naturally sedimentary soils (Mitchell, 1976), which encompasses all the effects that distinguish the mechanical properties of natural clay from remolded clay. Numerous studies have been conducted to evaluate the effects of soil structure experimentally or theoretically, from micro to macro point views (Leroueil *et al*., 1979; Locat and Lefebvre, 1986; Burland, 1990; Leroueil and Vaughan, 1990; Liu and Carter, 2000; Karstunen and Yin, 2010; Hong *et al*., 2006, 2007, 2012; Yin *et al*., 2011a, 2011b, 2011c). Generally, the compressibility of natural clay is significantly influenced by the soil structure. Before the yield stress, the compressibility of natural clay is restrained by the soil structure, resulting in relatively small deformation under consolidation stress. However, the soil structure is assumed to degrade after yielding, when the stress level exceeds the yield stress, the compressibility of natural clay increases dramatically due to the loss of initial soil structure. This state is referred as 'destructuration' (Leroueil *et al.*, 1979; Cotecchia and Chandler, 1997; Hong *et al.*, 2007). Hence, quantitatively evaluating the degradation of soil structure during destructuration is of great importance for both theoretical research and engineering practice (Burland, 1990; Hong, 2006).

*Lecturer, Geotechnical Research Institute, College of Civil and Transportation Engineering, Hohai University, Nanjing 210098, P.R. China; Key Laboratory of Ministry of Education for Geomechanics and Embankment Engineering, Hohai University, Nanjing 210098, P.R. China (Corresponding Author, E-mail: xia.bian@hhu.edu.cn)

**Associate Professor, Institute of Geotechnical Engineering, School of Transportation, Southeast University, Nanjing 210096, P.R. China (E-mail: jwding2006@163.com)

***Doctoral Student, Institute of Geotechnical Engineering, School of Transportation, Southeast University, Nanjing 210096, P.R. China (E-mail: scorpiojian@126.com)

****Doctoral Student, Institute of Geotechnical Engineering, School of Transportation, Southeast University, Nanjing 210096, P.R. China (E-mail: jasonyouyou@126.com)

Several theoretical frameworks for quantifying the effects of soil structure during compression have been proposed. Liu and Carter (2000) proposed that the additional void ratio Δe, referring to the difference of void ratio between natural and remolded clay, can be used as a measure of the influences of soil structure on compression behavior. Accordingly, a theoretical equation between additional void ratio Δe and the consolidation stress had been proposed to quantify the evolution of soil structure on the compression behavior. Alternatively, Cotecchia and Chandler (2000) and Chandler (2000) defined the stress sensitivity as the ratio of the yield stress of natural clay to the corresponding stress on the ICL, which can also be regarded as a parameter representing the differences of compression behavior between natural and remolded clay. However, the definition of stress sensitivity proposed by Chandler and his coworkers could only represent the initial degree of soil structure (Callisto and Rampello, 2004). Gasparre and Coop (2008) found that by extending the definition of stress sensitivity to current stress level at the post-yield state, the current stress sensitivity showed an increase tendency with the increase of stress level for stiff London clays, which failed to represent the degradation of soil structure during compression. Moreover, it is reported that the swell index of natural clay gradually converges to the swell index of remolded clay as destructuration occurring. Thus, the ratio of the intrinsic to the natural swelling index, defined as "swell sensitivity", can also be used to quantify the degradation of soil structure (Schmertmann, 1969; Picarelli, 1991; Burland *et al*., 1996). Although lots of parameters have been proposed to quantitatively assess the effects of soil structure on compression behavior of natural clay, little study at present can be found to quantitatively evaluate the degradation of soil structure using stress sensitivity or swell sensitivity for soft marine clay based on experimental data. Furthermore, the differences of the variation of soil structure during destructuration represented by different parameters (e.g., stress sensitivity, swell sensitivity and additional void ratio) also need to be further addressed.

The objective of this study is to investigate the variation of soil structure for natural soft marine clay during destructuration. A series of oedometer tests were carried out on undisturbed and remolded samples of Wenzhou marine clay. The unloading-reloading stages along the compression curves at different stress levels were also performed. Specifically, comparisons between the degradation law of soil structure represented by stress sensitivity and swell sensitivity, stress sensitivity and addition void ratio are investigated. This article also presents the correlation between the soil constant yielding the variation of stress sensitivity with natural void ratio and void ratio at liquid limit.

2. Tested Materials and Experimental Procedure

Two set of Wenzhou clays used in this study are typical marine deposit soft clay collected from Wenzhou city. They are designated as Louqiao clay and Ouhai clay, which are collected from Louqiao and Ouhai districts respectively, in Wenzhou, China. The undisturbed samples of Louqiao clay were retrieved with a thin-wall free-piston sampler by slowly pushing 80 mm diameter, 50 cm length polished stainless steel sharp edged sampler at designated depth. For the drilling at this site, a water pressure through a core tube is applied to remove the slimes between the designated depth for sampling. On the other hand, the undisturbed samples of Ouhai clay were retrieved as 50 cm × 50 cm × 50 cm block sample at the corresponding depth. Some physical properties (e.g., particle size composition, natural water content w_n, liquid limit w_L, plastic limit w_p, liquid index I_p and initial void ratio e_0) are summarized in Fig. 1. As reported by Yin *et al*. (2015), the upper 30 m depth Wenzhou soft clay layer is relative homogenous. It can be observed from Fig. 1 that the clay fraction, Atterberg limit and initial void ratio of Ouhai clay are consistent with that of Louqiao clay. The liquid limit and plastic limit for Louqiao and Ouhai clay ranges from 64.5% to 75.9% and 28% to 33.3% respectively. According to the Unified Soil Classification System (ASTM D 2487), the Louqiao and Ouhai clay can be classified as high plasticity clay (CH). The yield stress σ'_{vy} and in situ effective overburden stress σ'_{v0} are also

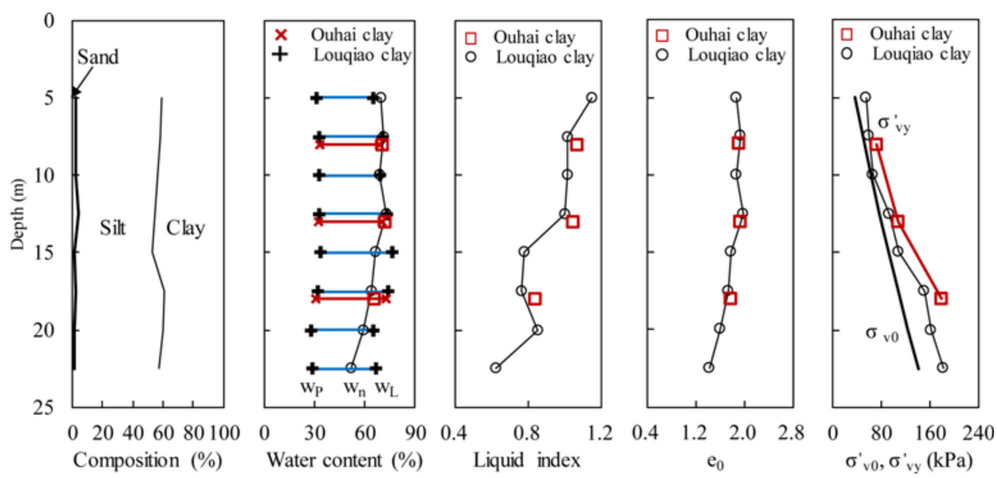

Fig. 1. Physical Properties of Wenzhou Clay: Particle Size Composition, Natural Water Content w_n, Liquid Limit w_L, Plastic Limit w_P, Liquid Index, Initial Void Ratio e_0 and Vertical Yield Stress σ'_{vy}.

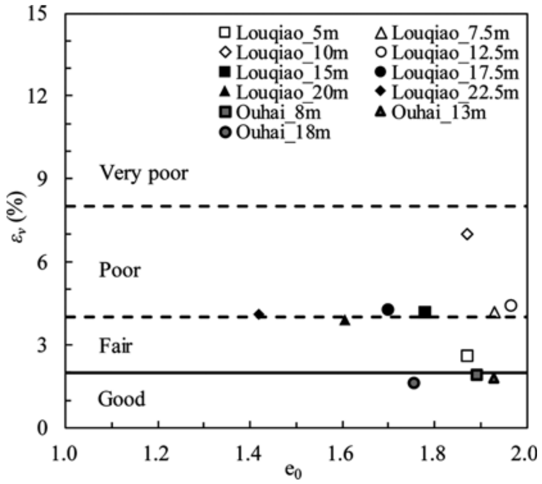

Fig. 2. Sample Quality Classification

shown in Fig. 1. It seems that the variation in yield stress of Louqiao clay is quite different from that of Ouhai clay. This behavior is mainly due to the different sampling methods used to obtain this two set of clays.

The quality of undisturbed samples are analyzed by measuring volumetric strain at the effective overburden stress from the compression curves (Lacasse *et al.*, 1985). Fig. 2 shows the sample quality classifications of the Louqiao and Ouhai clay. It can be conducted that the quality of most block samples are good (Ouhai clay), whereas the quality of most thin-wall samples are fair (Louqiao clay), except the sampling quality at depth of 10 m is relatively poor.

Table 1. Test Programme of Oedometer Test for Both Undisturbed and Remolded Samples

Samples Depth		Initial water content (%)	Consolidation stress (kPa)
Louqiao_5 m	undisturbed	69.4	6.25-12.5-25*-50*-100*-200-400*-800-1600*
Louqiao_7.5 m	undisturbed	70.8	
Louqiao_10 m	undisturbed	69.1	
Louqiao_12.5 m	undisturbed	72.9	
Louqiao_15 m	undisturbed	66.2	
Louqiao_17.5 m	undisturbed	63.6	
Louqiao_20 m	undisturbed	59.2	
Louqiao_22.5 m	undisturbed	52.0	
Ouhai_8 m	undisturbed	70.1	
Ouhai_13 m	undisturbed	71.4	
Ouhai_18 m	undisturbed	65.2	
Louqiao_5 m	remolded	67.2	0.5-1.5-2.5-4.5-8.5-12.5-25-50-.100*-200*-400*-800-1600*
Louqiao_7.5 m	remolded	70.7	
Louqiao_10 m	remolded	71.0	
Louqiao_12.5 m	remolded	73.4	
Louqiao_15 m	remolded	65.3	
Louqiao_17.5 m	remolded	64.2	
Louqiao_20 m	remolded	59.4	
Louqiao_22.5 m	remolded	52.6	
Ouhai_8 m	remolded	70.5	
Ouhai_13 m	remolded	71.7	
Ouhai_18 m	remolded	65.0	

Note: *The swelling tests were performed on the specimens at the marked load increments.

A series of oedometer tests on both undisturbed and remolded samples of Wenzhou clay were performed to study the difference in compression behavior between these two kinds of samples. The rest of soil from undisturbed samples were collected and remolded by a mixer at their natural water content to create a homogenous soil paste. Then, the remolded soil paste was putted into the oedometer ring layer by layer to form the remolded sample. Finally, the measured values of mass of remolded samples were compared with that of corresponding undisturbed samples to assess the sample quality. It is found that the error of all remolded samples is less than 1%. The modified one-dimensional consolidation apparatus starting from a low vertical effective stress (0.5 kPa) was adopted to perform the oedometer tests for remolded clay (Hong *et al*., 2010). The loading steps for the remolded specimens were 0.5 kPa, 1.5 kPa, 2.5 kPa, 4.5 kPa, 8.5 kPa, 12.5 kPa, 25 kPa, 50 kPa, 100 kPa, 200 kPa, 400 kPa, 800 kPa and 1600 kPa. The conventional oedometer apparatus was used for undisturbed specimens and the loading steps change from 6.25 kPa to 1600 kPa by doubling the load for each increment. All soil specimens have a diameter of 61.8 mm and a height of 20 mm. The duration of every load increment is about 24 h. The unloading-reloading tests were performed on both undisturbed and remolded clay at different stress level to investigate the influence of soil structure and stress level on swell index. Detailed test programme is listed in Table 1.

3. Test Results

3.1 Compression Curves

Figure 3 shows the compression curves of undisturbed and remolded samples for Wenzhou marine clay. The symbol U and R represent the undisturbed and the remolded samples respectively in Fig. 3. The number after U and R represent the depth of retrieval (in meters). It can be seen from Fig. 3 that the compression curves of all undisturbed samples lie above that of corresponding remolded samples, and show a typical inverse ‘S’ shape as a result of the effects of soil structure. The yield stress can be clearly identified in all compression curves of undisturbed samples. When the consolidation stress is lower than the yield stress, small compressibility occurs due to the soil structure during depositional and post-depositional processes (Leroueil and Vaughan, 1990; Cotecchia and Chandler, 1997). After yielding, the compressibility of natural clay increases dramatically as a result of gradually breakdown of soil structure.

Meanwhile, it is noteworthy that when the oedometer test is starting from a very small stress level (e.g., 0.5 kPa in this study), the compression curves of remolded clay also show an inverse ‘S’ shape, similar to that of undisturbed clay. Hong *et al.* (2010, 2012) pointed that this phenomenon is resulted from the remolded yield stress (σ'_{yr}). The compressibility of remolded clay is small in the case of $\sigma'_v < \sigma'_{yr}$ and the compressibility

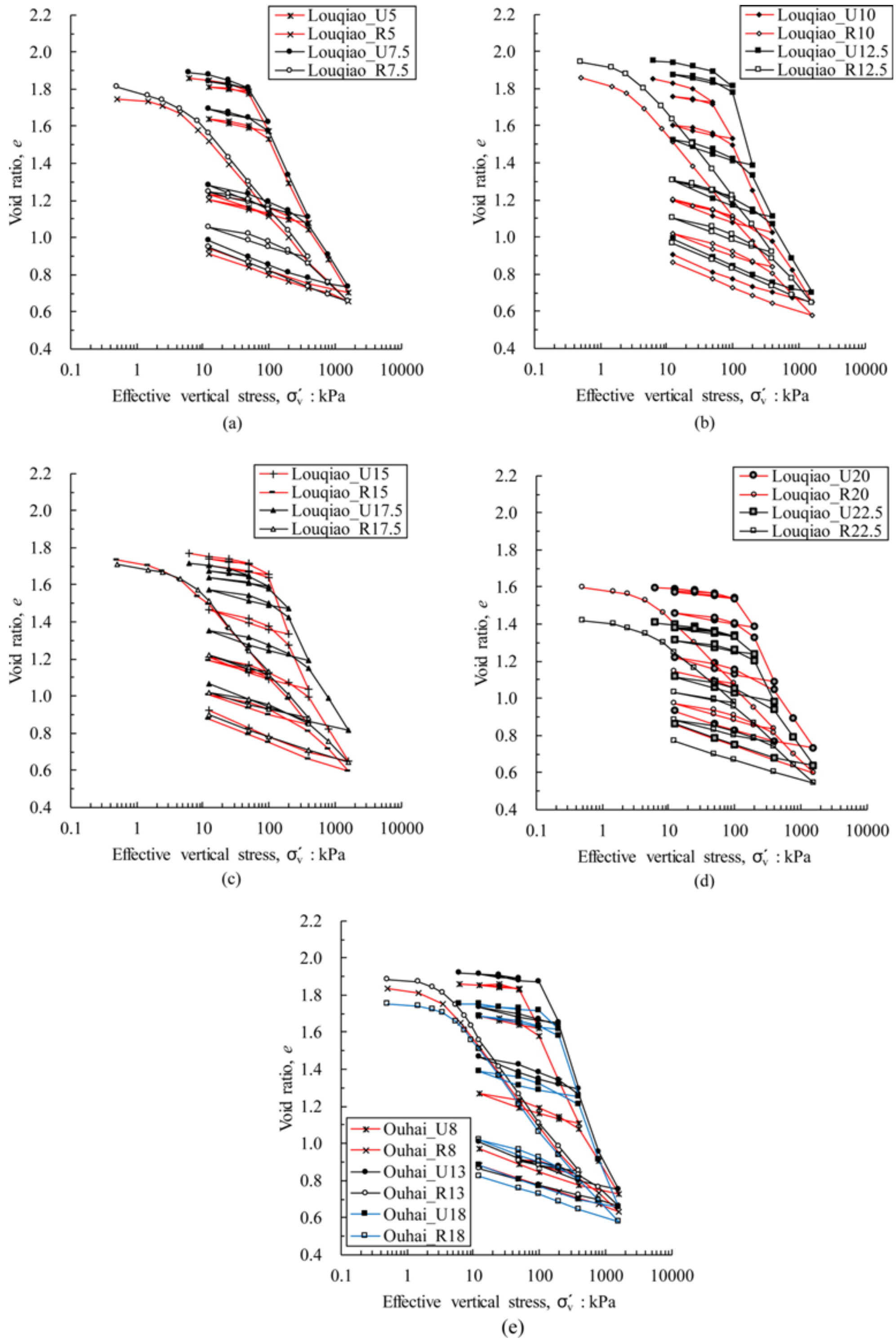

Fig. 3. Compression Curves of Natural and Remolded Clay: (a) Louqiao Clay (depth = 5 m and 7.5 m), (b) Louqiao Clay (depth = 10 m and 12.5 m), (c) Louqiao Clay (depth = 15 m and 17.5 m), (d) Louqiao Clay (depth = 20 m and 22.5 m), (e) Ouhai Clay (depth = 8 m, 13 m and 18 m)

dramatically increases when the stress level exceeds σ'_{yr}. Hence, as the starting point of conventional oedometer test is about 10 kPa according to common standards (e.g., ASTM D 2435, BS 1377), the initial part of compression curves ($\sigma'_v \leq 10$ kPa) will be missing. Consequently, the compression curves of remolded clay may be seemed as a slightly concave line as described by Burland (1990) when conventional oedometer test starts from 10 kPa.

3.2 Normalized Compression Curves

To emphasize the soil structure of natural clay, Burland (1990) introduced void index (I_v) to normalize the compression curves of remolded clay. The void index I_v is defined as

$$I_v = (e - e^*_{100})/(e^*_{100} - e^*_{1000}) = (e - e^*_{100})/C^*_C \quad (1)$$

where e^*_{100} and e^*_{1000} are the void ratios of remolded clay at the effective vertical stresses of 100 kPa and 1000 kPa, respectively, C^*_C is the intrinsic compression index. Burland (1990) proposed the Intrinsic Compression Line (ICL) to normalize the compression curves of remolded clay with initial water contents of 1.0-1.5 times the liquid limits, when the effective vertical stress varied from 10 kPa to 4000 kPa, expressed as:

$$I_v = 2.45 - 1.285\log\sigma'_v + 0.015(\log\sigma'_v)^3 \quad (2)$$

where σ'_v is expressed in kPa.

Figure 4 shows the normalized compression curves of remolded and natural Wenzhou clay. The unloading-reloading curves are removed for clarity in Fig. 4. The ICL and the Sedimentation Compression Line (SCL) are also shown in the same figure for comparison. The Sedimentation Compression Line (SCL) is expressed in terms of the in situ void index, I_{v0}, against the effective overburden stress, σ'_{v0}. It is evident from Fig. 4(a) that when the stress level is higher than the remolded yield stress, the compression curves of remolded clay can be normalized well by the ICL as suggested by Hong *et al*. (2010). Fig. 4(b) illustrates that all the normalized compression curves of natural clay lie above the ICL. It is also found that the yielding of all Wenzhou marine clay lies to the right of the SCL, which is an indicative of significant bonding (Burland *et al*., 1996). It should be emphasized that the compression curve of natural clay is substantially affected by soil disturbance during sampling (Chung *et al*., 2004; Sun *et al*., 2014). The compression curve of Ouhai clay with slight disturbance lies to the right of the Louqiao clay at the same depth with relatively large disturbance, resulting in higher yield stress as shown in Fig. 1. Hence, it is of practically importance to

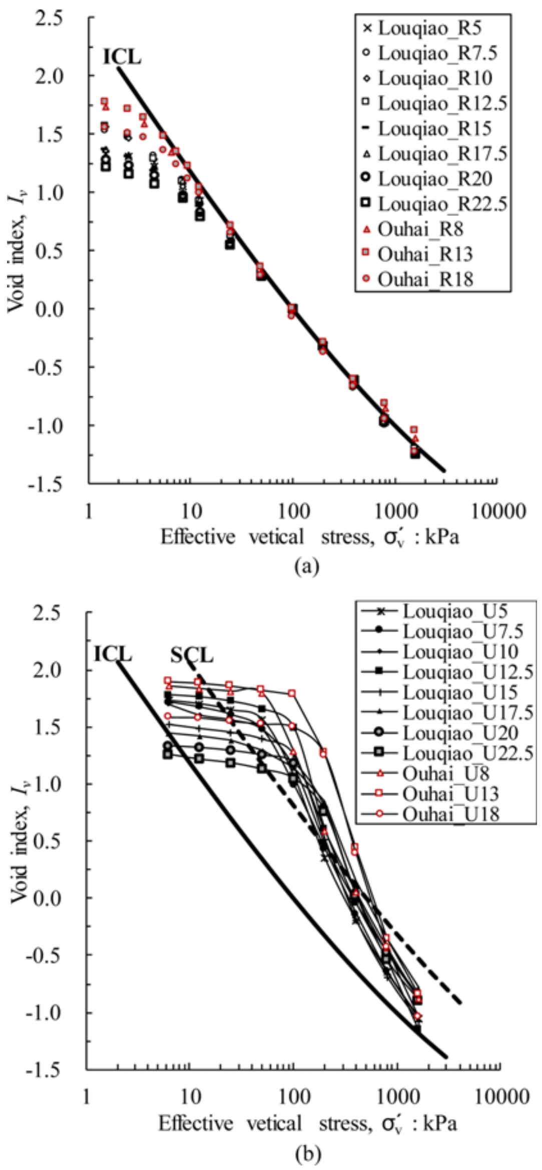

Fig. 4. Void Index Versus Effective Vertical Stress: (a) Remolded Clay, (b) Undisturbed Clay

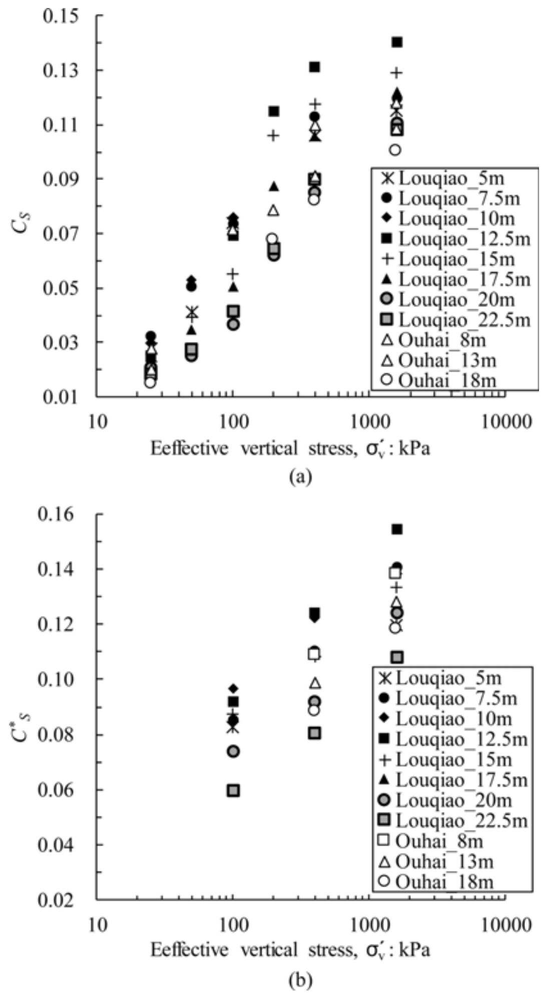

Fig. 5. Swell Index Versus Effective Vertical Stress for: (a) Natural, (b) Remolded Wenzhou Clay

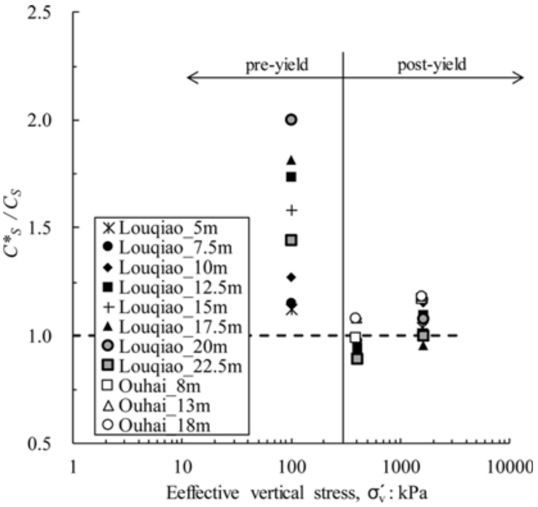

Fig. 6. Change of C_s^*/C_s at Different Stress Level

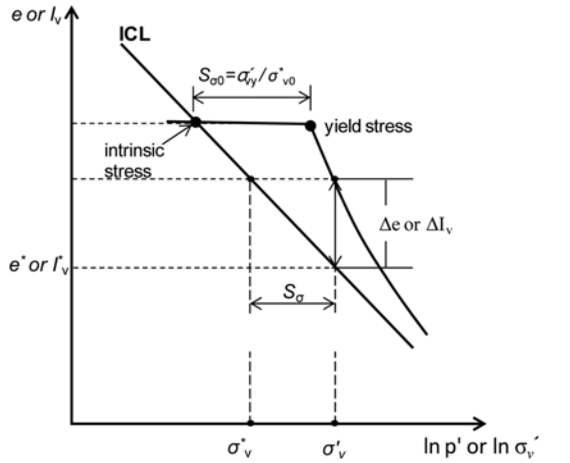

Fig. 7. Schematic Diagram of Stress Sensitivity

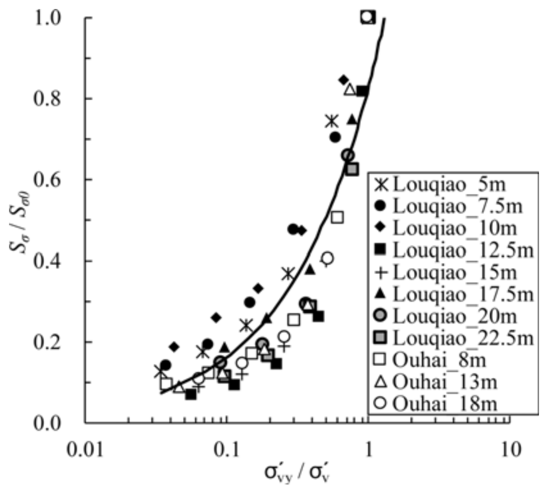

Fig. 8. Change of Normalized Stress Sensitivity with the Consolidation Stress

minimize the disturbance of undisturbed clay during sampling, transporting and testing.

3.3 Swell Index Versus Stress Level

The swell index is defined as the average slop of unloading-reloading curve in the semi-logarithm plot as shown in Fig. 3. The variation of swell index with the consolidation stress for undisturbed and remolded clay is plotted in Fig. 5(a) and (b) respectively. The effective vertical stress in Fig. 5 represents the stress level when the unloading starts. It can be seen from Fig. 5 that the swell index C_s of natural clay increases steadily with an increase in consolidation stress due to the breakdown of soil structure during destructuring (Picarelli, 1991). Meanwhile, it is interesting to note that the swell index C_s^* of remolded clay also increases with the consolidation stress. This behavior implies that the variation of swell index C_s for the natural clay is partly due to the breakdown of soil structure, and partly due to the increase of stress level. Thus, it is logical to compare the swell index of natural clay with that of corresponding remolded clay at the same stress level. It can be observed from Fig. 6 that when the stress level is lower than the yield stress, C_s of natural clay is smaller than that of corresponding remolded clays due to the effects of soil structure (Schmertmann, 1969; Gasparre and Coop, 2008). While the stress level exceeds the yield stress, the swell index of natural clay is almost identical to that of remolded clay (i.e., C_s^*/C_s = 0.89-1.17). This confirms that the bonding effects of natural Wenzhou marine clay are already breakdown in the post-yield regime (Burland *et al.*, 1996).

4. Quantifying Soil Structure During Destructuration

4.1 Comparison between Stress Sensitivity and Swell Sensitivity During Destructuration

Stress sensitivity proposed by Chandler (2000) and Cotecchia and Chandler (2000), is often used as a quantitative parameter for evaluating the effects of soil structure. Fig. 7 shows the schematic diagram of stress sensitivity. The ratio of the yield stress of natural clay to the vertical stress on the ICL at the same void ratio ($\sigma'_{vy}/\sigma^*_{v0}$) is defined as 'stress sensitivity'. Callisto and Rampello (2004) pointed out that this definition was only accounting for the initial degree of soil structure, however, did not account for a continuous change in structure during destructuration. Hence, a current S_σ can be defined as the ratio of the current vertical stress between the natural clay and the ICL at the same void index (Gasparre and Coop, 2008), expressed as,

$$S_\sigma = \frac{\sigma'_v}{\sigma^*_v} \tag{3}$$

where σ'_v and σ^*_v represent the current stress level of the natural clay and its ICL at the same void index, respectively.

Due to the influence of soil disturbance on the mechanical behavior of natural clay, the current stress sensitivity S_σ is normalized by stress sensitivity at yield stress $S_{\sigma 0}$, termed as normalized stress sensitivity $S_\sigma/S_{\sigma 0}$, to investigate the variation

Table 2. Parameters α and β for Wenzhou clay

Depth	α	β	ΔI_{vi}	b
Louqiao_5 m	0.96	0.63	1.32	0.55
Louqiao_7.5 m	1.00	0.61	1.09	0.51
Louqiao_10 m	0.99	0.53	0.86	0.42
Louqiao_12.5 m	0.86	0.94	1.43	0.82
Louqiao_15 m	0.93	0.80	1.18	0.80
Louqiao_17.5 m	0.90	0.61	1.14	0.51
Louqiao_20 m	0.85	0.66	1.31	0.60
Louqiao_22.5 m	0.99	0.74	1.04	0.66
Ouhai_8 m	1.09	0.75	1.30	0.68
Ouhai_13 m	0.94	0.94	1.60	0.85
Ouhai_18 m	0.88	1.00	2.00	0.90

of soil structure during destructuration. Fig. 8 depicts the variation of normalized stress sensitivity with the consolidation stress. It is evident from Fig. 8 that current S_σ of natural clay monotonically decreases with an increase in consolidation stress, which is consistent with the progressively loss of the soil structure after yielding. Hence, current S_σ can be used as the quantification for assessing the degradation of soil structure of soft marine clay. Moreover, normalized stress sensitivity $S_\sigma/S_{\sigma 0}$ has a good parabolic relationship with the consolidation stress σ'_v , expressed as

$$\frac{S_\sigma}{S_{\sigma 0}} = \alpha\left(\frac{\sigma'_{vy}}{\sigma'_v}\right)^\beta \tag{4}$$

where σ'_{vy} represents the yield stress, the parameter α and β are the soil constants. The correlation coefficient (R^2) for Wenzhou marine clay investigated are from 0.92 to 0.97, indicating that current S_σ can be sufficiently used to quantify the difference of compression behavior between natural and remolded clay. Table 2 shows the values of α and β for Wenzhou clay. It is interesting to note that the parameter α ranges from 0.83 to 1.18, with an average value of 0.97 for Wenzhou clay in this study and 50 undisturbed clays from literature (as shown in Table 3). Hence, Eq. (4) can be simplified as following by assuming $\alpha = 1$

$$S_\sigma = S_{\sigma 0}\left(\frac{\sigma'_{vy}}{\sigma'_v}\right)^\beta \tag{5}$$

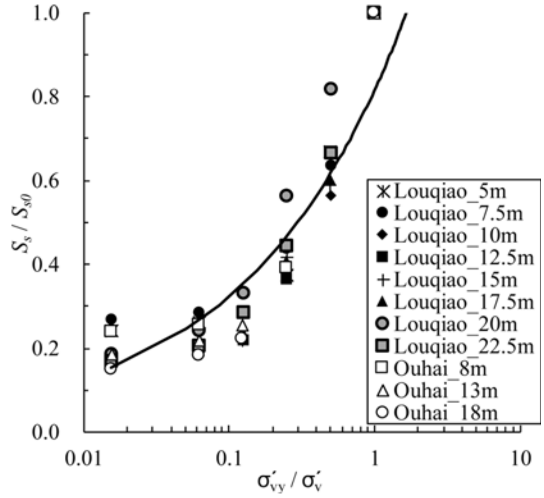

Fig. 9. Change of Normalized Swell Sensitivity with the Consolidation Stress

Swell sensitivity proposed by Schmertmann (1969) is defined as the ratio of the intrinsic to the undisturbed swelling index ($S_s = C_s^*/C_s$). However, it has been found that the swell index of remolded clay also varies with the consolidation stress, then the swell index of remolded clay at unloading pressure of 1600 kPa (C_{s1600}^*) in this study is chosen as the reference value for calculating swell sensitivity to eliminate the influence of stress level. The variation of normalized swell sensitivity S_s/S_{s0} with the consolidation stress is plotted in Fig. 9. where S_{s0} represents the swell sensitivity of natural clay at the pre-yield regime. The effective vertical stress in Fig. 9 is the consolidation stress prior

Table 3. Sources of Natural Clays from Literature

Clay	Depth (m)	G_s (g/cm^3)	w_n (%)	w_P (%)	w_L (%)	e_L	e_0	α	β	Source of data
Vasby clay	4.3	2.65	124	39	122.5	3.25	3.29	0.97	1.22	Kabbaj *et al.* (1988)
Bothkennar clay	5	2.80	70	41	85	2.38	1.96	0.96	0.37	Hight *et al.* (1992)
Sault Ste Marie Clay	6.55-9.75	2.65	50.251.5	-	47.0-48.1	1.25-1.27	1.33-1.36	0.97-1.01	0.85-0.97	Burland (1990)
Shellhaven clay	7.5-10.4	2.66	78.5-82.1	-	72-85	1.92-2.26	2.09-2.18	0.96-0.97	0.61-0.68	Burland (1990)
Toll field clay	6.5-21.2	2.68	53.7-53.9	-	54.6-69.2	1.46-1.85	1.44	0.96-0.99	0.54-0.62	Burland (1990)
Pusan clay	-	2.63-2.73	41-100	14-28	37-69	1.27-1.82	1.20-2.26	0.87-1.18	0.53-1.09	Chung *et al*. (2004)
Fukuoka clay	11.4-13.4	2.69-2.71	56.1-70.1	34.7-44.9	70-83	1.88-2.25	1.51-1.9	0.92-0.98	0.12-0.46	Hong *et al.* (2007)
Ariake clay	3.4-8.4	2.64-2.68	86.3-97.5	32.5-36.4	73.9-84.5	1.96-2.25	2.29-2.59	0.86-0.90	0.70-1.15	Hong *et al.* (2007)
Bangkok clay	3-17	2.64	47-84	23-26	58-88	1.53-2.32	1.24-2.22	0.83-1.18	0.64-0.73	Horpibulsuk *et al.* (2007)
Singapore clay	25–41	2.74-2.79	55.3-67.6	-	82–92	2.05-2.36	1.53-1.86	0.93-1.00	0.41-0.72	Low *et al*. (2008)
Osaka Bay clay	73	2.66	69	38	100	2.66	1.84	0.97	0.32	Watabe *et al.* (2008)
Murro clay	7	2.70	89	35.8	92	2.48	2.40	0.95	0.54	Karstunen and Yin (2010)
Lianyungang clay	4-12	2.72-2.74	67.9-80.1	30-30.9	62.7-86	1.71-2.36	1.85-2.19	0.96-1.02	0.3-0.54	Zeng *et al.* (2011)
Nanjing clay	7-9	2.7-2.72	42-46.8	22.8-25.9	42-43.8	1.18-1.41	1.13-1.27	1.01-1.02	0.32-0.62	Zeng *et al*. (2011)
Wenzhou clay	11	2.80	63	27.5	67	1.88	1.76	0.98	0.62	Yin *et al.* (2015)

to the start of the unloading stage. It is interesting to note that the variation of swell sensitivity with the consolidation stress has the same tendency as that of stress sensitivity.

Comparing Fig. 8 and Fig. 9, it can be conducted that although the variation of swell sensitivity during destructuration is similar to that of stress sensitivity for natural Wenzhou marine clay, the swell sensitivity might only be regarded as a qualitative parameter due to the fact that the swell index varying with the stress level for remolded clay.

4.2 Comparison between Stress Sensitivity and Additional Void Ratio During Destructuration

One important method to quantify the degradation of soil structure during compression is the theoretical formula proposed by Liu and Carter (2000), expressed as the change of additional void ratio with the consolidation stress,

$$\Delta e = \Delta e_i\left(\frac{\sigma'_{vy}}{\sigma'_v}\right)^b \tag{6}$$

where Δe is the additional void ratio, representing the difference of void ratio between natural and remolded clay at the same stress level, as schematically shown in Fig. 7, Δe_i is the additional void ratio at the yield stress, b is the compression destructuring index.

Comparing Eq. (5) with (6), it is found that the variation of soil structure with the consolidation stress in terms of stress sensitivity or additional void ratio is quite similar. Hence, it is important to evaluate the relationship between $S_{\sigma 0}$ and Δe_i, or β and b, quantitatively assessing the degradation law of soil structure in terms of different parameters.

To investigate the relationship between $S_{\sigma 0}$ and Δe_i, or β and b, conventional consolidation test data of other natural clays were compiled from the available literature. Table 3 summarizes the available results of oedometer tests on 50 undisturbed clays. Fig. 10 presents the relationship between $S_{\sigma 0}$ and ΔI_{vi}, where ΔI_{vi} represents the additional void index at the yield stress, defined as

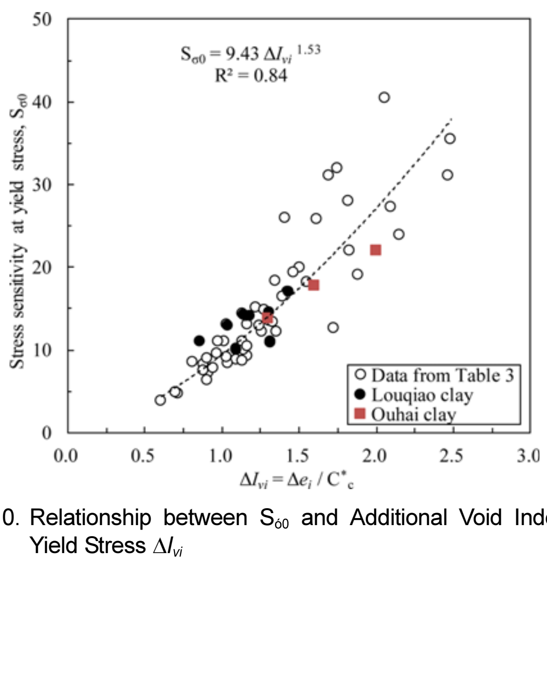

Fig. 10. Relationship between $S_{\sigma 0}$ and Additional Void Index At Yield Stress ΔI_{vi}

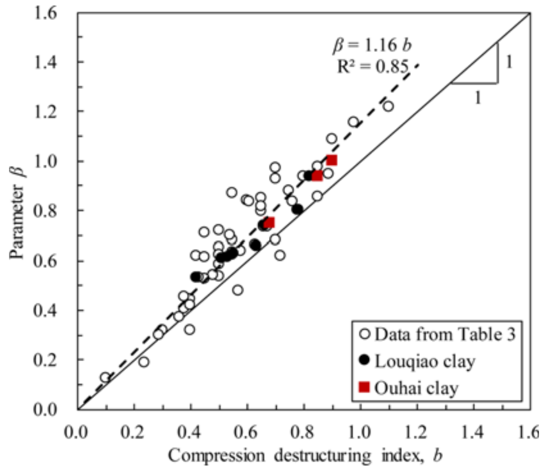

Fig. 11. Relationship between Parameter β and Compression Destructuring Index b

$\Delta e_i/C^*_c$. Note that the intrinsic parameter C^*_c were calculated based on the empirical equation proposed by Burland (1990), expressed as $C^*_c = 0.256e_L - 0.04$, where e_L represents the void ratio at liquid limit. It can be observed from Fig. 10 that the relationship between $S_{\sigma 0}$ and ΔI_{vi} can be expressed, with a correlation coefficient ($R^2 = 0.84$) as

$$S_{\sigma 0} = 9.43(\Delta e_i/C^*_c)^{1.53} \tag{7}$$

Hong (2006) suggested that ΔI_{vi} is an indication of soil structure. The clay with a relatively higher position compared with the ICL (higher ΔI_{vi}) has a larger stress sensitivity, possesses a stronger soil structure. Based on the increase tendency of $S_{\sigma 0}$ with ΔI_{vi} from Equation (6), it can be conducted that the clay with a higher $S_{\sigma 0}$ possesses a stronger soil structure. Hence, the low value of $S_{\sigma 0}$ for Louqiao clay may be due to the soil disturbance during sampling (Hong, 2006).

Figure 11 presents the correlation between β and the compression destructuring index b. It can be observed that β presents a linear relationship with b. Note that the slope of this linear formula is 1.16, which is close to unit. This implies that the degradation rate of soil structure during compression in terms of additional void ratio and stress sensitivity is almost identical. Comparing Eq. (5) with Eq. (6), it can be deduced that,

$$\Delta e/S_\sigma \approx \Delta e_i/S_{\sigma 0} \tag{8}$$

Equation (8) suggests that during compression, additional void ratio is an indicator to normalize stress sensitivity. Hence, it can be conducted that the variation of soil structure during destructuration represented by stress sensitivity and additional void ratio is essentially the same.

5. Correlations of β with Atterberg Limits

It is common practice to correlate important soil constants (e.g., undrained strength, compressibility or creep coefficient) with some basic engineering properties of soils (e.g., Atterberg

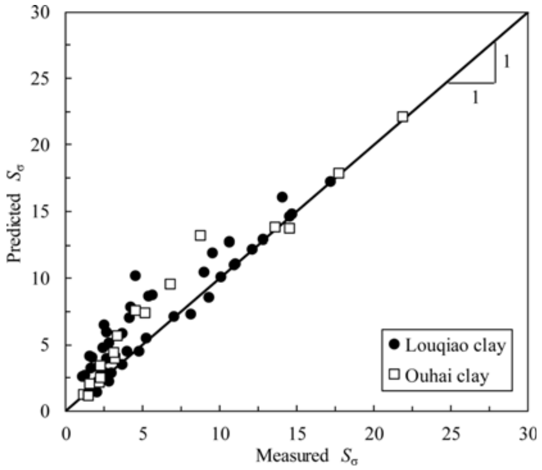

Fig. 12. Comparison between the Stress Sensitivity Predicted and Oedometer Test Data for Wenzhou Clay in this Study

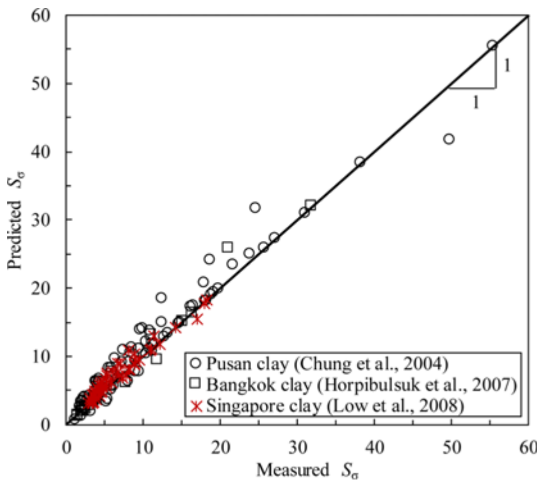

Fig. 13. Comparison between the Stress Sensitivity Predicted and Oedometer Test Data for: Pusan Clay Reported by Chung *et al.* (2004); Bangkok Clay Reported by Horpibulsuk *et al.* (2007) and Singapore Clay Reported by Low *et al.* (2008)

limits), which can reduce some time-consuming experimental procedure and provide rational first-order estimation of these important soil properties during planning stage or in engineering practice (Wroth and Muir Wood, 1978; Al-Khafaji and Andersland, 1992; Yoon and Kim, 2006; Zhu *et al.*, 2015; Zeng *et al.*, 2015). Hence, it is of practical importance to correlate the material constants relating to the degradation of soil structure with Atterberg limits.

For correlating β with Atterberg limits, the data from the 50 oedometer tests in Table 3 together with the experimental data on 11 Wenzhou clays in this study were used to perform a multiple regression analysis. It is found that the parameter β is correlated to two simple physical parameters: natural void ratio (e_0) and void ratio at liquid limit (e_L). The optimized correlation of β can be expressed as follows:

$$\beta = 1.74 - 0.45e_L + 0.026e_L^2 - 0.69e_0 + 0.26e_0^2 \quad (9)$$

The values of multiple regression coefficients (R^2) is 0.55 for Eq. (9).

To this end, if the initial degree of soil structure ($S_{\sigma 0}$), natural void ratio (e_0) and void ratio at liquid limit (e_L) of the natural clay are known, the current S_σ can be determined with the following steps: 1) determining β in Eq. (9); 2) determining the initial degree of soil structure $S_{\sigma 0}$ from oedometer test or unconfined compression test; 3) determining the variation of stress sensitivity S_σ with the consolidation stress in Eq. (5). For assessing the relationship between current S_σ predicted by Eq. (5) and the experimental data, comparisons for Wenzhou clay in this study and for selected clays from Table 3 are shown in Fig. 12 and Fig. 13 respectively. Despite of certain discrepancies between measured and predicted values; the proposed empirical equation generally describes the evolution of stress sensitivity S_σ fairly well.

6. Conclusions

This paper presents an investigation into the variation of soil structure during compression for Wenzhou marine clay by conducting a series of oedometer tests. The main conclusions obtained from this paper are summarized as follows:

1. The swell index for both natural and remolded Wenzhou clay increases with the increase in stress level. The swell index of natural clay is smaller than that of remolded clay due to the effects of soil structure when the stress level is lower than the yield stress. While the stress level exceeds the yield stress, which is sufficient to destroy the bonding effects of natural Wenzhou marine clay, the swell index of natural clay is almost identical to that of remolded clay.
2. The stress sensitivity is proven to be an efficient quantification of the degradation of soil structure for Wenzhou marine clay. Normalized stress sensitivity $S_\sigma/S_{\sigma 0}$ has a good parabolic relationship with the consolidation stress for natural Wenzhou marine clay. Due to the variation of swell index with the stress level for remolded clay, the swell sensitivity can only be regarded as a qualitative parameter.
3. Stress sensitivity at yield stress $S_{\sigma 0}$ correlates well with the additional void index at yield stress ΔI_{vi}. Meanwhile, the parameter β representing the degradation rate of soil structure in the relationship between the stress sensitivity and the consolidation stress, is almost identical to the compression destructuring index b. This indicates that the variation of soil structure during destructuration represented by stress sensitivity and additional void ratio are essentially the same.
4. The correlation between the soil constant β and the natural void ratio, the void ratio at liquid limit is proposed and can be determined by following equation: $\beta = 1.74\text{-}0.45\ e_L + 0.026e_L^2 - 0.69e_0 + 0.26e_0^2$.

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Notations

b = Compression destructuring index
C_s = Swell index for natural clay
C_s^* = Swell index for remolded clay
e = Void ratio
e^* = Void ratio of remolded clay
e_L = Void ratio at liquid limit
e_0 = Natural void ratio
e_{100}^* = Void ratio of remolded clay at σ'_v = 100 kPa
e_{1000}^* = Void ratio of remolded clay at σ'_v = 1000 kPa
G_s = Density of soil particles
ICL = Intrinsic compression line
SCL = Sedimentary compression line
S_σ = Stress sensitivity
$S_{\sigma 0}$ = Stress sensitivity at the yield stress
S_s = Swell sensitivity
w_0 = Initial water content
w_L = Liquid limit
w_P = Plastic limit
w_n = Natural water content
Δe = Additional void ratio between natural clay and remolded clay
Δe_i = Additional void ratio at yield stress
σ'_v = Vertical effective stress
σ'_{v0} = Vertical overburden stress
σ'_{vy} = Vertical yield stress for natural clay
σ'_{yr} = Remolded yield stress
σ_v^* = Vertical effective stress at ICL
I_v = Void index, equal to $(e - e_{100}^*)/(e_{100}^* - e_{1000}^*)$
I_v^* = Void index of remolded clay
ΔI_{vi} = Additional void index at yield stress, equal to $\Delta e_i / C_c^*$

References

Al-Khafaji, A. W. N. and Andersland, O. B. (1992). "Equations for compression index approximation." *Journal of Geotechnical Engineering*, Vol. 118, No. 1, pp. 148-153, DOI: 10.1061/(ASCE) 0733-9410(1992)118:1(148).

ASTM D2435/D2435M-11 (2011). *Standard test methods for one-dimensional consolidation properties of soils using incremental loading*, ASTM International, West Conshohocken, PA.

ASTM D2487-11 (2011). *Standard practice for classification of soils for engineering purposes (unified soil classification system)*, ASTM International, West Conshohocken, PA.

British Standard, BS 1377 (1990). *British standard methods of test for soils for civil engineering purposes*, British standard institution, London.

Burland, J. B. (1990). "On the compressibility and shear strength of natural soils." *Géotechnique*, Vol. 40, No. 3, pp. 329-378, DOI: 10.1680/geot.1990.40.3.329.

Burland, J. B., Rampello, S., Georgiannou, V. N., and Calabresi, G. (1996). "A laboratory study of the strength of four stiff clays." *Géotechnique*, Vol. 46, No. 3, pp. 491-514, DOI: 10.1680/geot.1996.46.3.491.

Callisto, L. and Rampello, S. (2004). "An interpretation of structural degradation for three natural clays." *Canadian Geotechnical Journal*, Vol. 41, No. 3, pp. 392-407, DOI: 10.1139/T03-099.

Chandler, R. J. (2000). "The third glossop lecture: Clay sediments in depositional basins: The geotechnical cycle." *Quarterly Journal of Engineering Geology and Hydrogeology*, Vol. 33, No. 1, pp. 7-39, DOI: 10.1144/qjegh.33.1.7.

Chung, S. G., Prasad, K. N., Nagaraj, T. S., Chung, J. G., and Jo, K. Y. (2004). "Assessment of compressibility behavior of naturally cemented soft clays." *Marine Georesources and Geotechnology*, Vol. 22, Nos. 1-2, pp. 1-20, DOI: 10.1080/10641190490466892.

Chung, S. G., Ryu, C. K., Min, S. C., Lee, J. M., Hong, Y. P., and Odgerel, E. (2012). "Geotechnical characterisation of busan clay." *KSCE Journal of Civil Engineering*, Vol. 16, No. 3, pp. 341-350, DOI: 10.1007/s12205-012-1433-8.

Cotecchia, F. and Chandler, R. J. (1997). "The influence of structure on the pre-failure behaviour of a natural clay." *Géotechnique*, Vol. 47, No. 3, pp. 523-544, DOI: 10.1680/geot.1997.47.3.523.

Cotecchia, F. and Chandler, R. J. (2000). "A general framework for the mechanical behaviour of clays." *Géotechnique*, Vol. 50, No. 4, pp. 431-447, DOI: 10.1680/geot.2000.50.4.431.

Gasparre, A. and Coop, M. R. (2008). "Quantification of the effects of structure on the compression of a stiff clay." *Canadian Geotechnical Journal*, Vol. 45, No. 9, pp. 1324-1334, DOI: 10.1139/T08-052.

Hight, D. W., Bond, A. J., and Legge, J. D. (1992). "Characterization of the Bothkennaar clay: An overview." *Géotechnique*, Vol. 42, No. 2, pp. 303-347, DOI: 10.1680/geot.1992.42.2.303.

Hong, Z. S. (2006). "Correlating compression properties of sensitive clays using void index." *Géotechnique*, Vol. 56, No. 8, pp. 573-577, DOI: 10.1680/geot.2006.56.8.573.

Hong, Z. S., Shen, S. L., Deng, Y., and Negami, T. (2007). "Loss of soil structure for natural sedimentary clays." *Geotechnical Engineering, The Proceedings of ICE*, Vol. 160, No. 3, pp. 153-159, DOI: 10.1680/geng.2007.160.3.153.

Hong, Z. S., Yin, J., and Cui, Y. J. (2010). "Compression behaviour of remolded soils at high initial water contents." *Géotechnique*, Vol. 60, No. 9, pp. 691-700, DOI: 10.1680/geot.09.P.059.

Hong, Z. S., Zeng, L. L., Cui, Y. J., Cai, Y. Q., and Lin, C. (2012). "Compression behaviour of natural and remolded clays." *Géotechnique*, Vol. 62, No. 4, pp. 291-301, DOI: 10.1680/geot.10.P.046.

Horpibulsuk, S., Shibuya, S., Fuenkajorn, K., and Katkan, W. (2007). "Assessment of engineering properties of Bangkok clay." *Canadian Geotechnical Journal*, Vol. 44, No. 2, pp. 173-187, DOI: 10.1139/ t06-101.

Kabbaj, M., Tavenas, F., and Leroueil, S. (1988). "In situ and laboratory stress–strain relationships." *Géotechnique*, Vol. 38, No. 1, pp. 83-100, DOI: 10.1680/geot.1988.38.1.83.

Karstunen, M. and Yin, Z. Y. (2010). "Modelling time-dependent behaviour of Murro test embankment." *Géotechnique*, Vol. 60, No. 10, pp. 735-749, DOI: 10.1680/geot.8.P.027.

Kishore, Y. N., Rao, S. N., and Mani, J. S. (2009). "The Behavior of Laterally Loaded Piles Subjected to Scour in Marine Environment." *KSCE Journal of Civil Engineering*, Vol. 13, No. 6, pp. 403-406, DOI: 10.1007/s12205-009-0403-2.

Kulkarni, M. P., Patel, A., and Singh, D. N. (2010). "Application of shear wave velocity for characterizing clays from coastal regions." *KSCE Journal of Civil Engineering*, Vol. 14, No. 3, pp. 307-321, DOI: 10.1007/s12205-010-0307-1.

Lacasse, S., Berre, T., and Lefebvre. G. (1985). "Block sampling of sensitive clays." *Proceeding of 11th International Conference on Soil Mechanics and Foundation Engineering,* San Francisco, pp. 887-892.

Leroueil, S. and Vaughan, P. R. (1990). "The general and congruent effects of structure in natural soils and weak rocks." *Géotechnique*, Vol. 40, No. 3, pp. 467-488, DOI: 10.1680/geot.1990.40.3.467.

Leroueil, S., Tavenas, F., Brucy, F., La Rochelle, P., and Roy, M. (1979). "Behaviour of destructured natural clays." *Journal of Geotechnical Engineering, ASCE*, Vol. 105, No. 6, pp. 759-778.

Liu, M. D. and Carter, J. P. (2000). "Modelling the destructuring of soils during virgin compression." *Géotechnique*, Vol. 50, No. 4, pp. 479-483, DOI: 10.1680/geot.2000.50.4.479.

Locat, J. and Lefebvre, G. (1986). "The origin of structuration of the Grande-Baleine marine sediments, Québec, Canada." *Quarterly Journal of Engineering Geology and Hydrogeology*, Vol. 19, No. 4, pp. 365-374, DOI: 10.1144/GSL.QJEG.1986.19.04.03.

Low, H. E., Phoon, K. K., Tan, T. S., and Leroueil, S. (2008). "Effect of soil microstructure on the compressibility of natural Singapore marine clay." *Canadian Geotechnical Journal*, Vol. 45, No. 2, pp. 161-176, DOI: 10.1139/T07-075.

Mitchell, J. K. (1976). *Fundamentals of soil behavior*, New York: Wiley.

Picarelli, L. (1991). "Discussion on the paper "the general and congruent effects of structure in natural soils and weak rocks" by Leroueil and Vanghan." *Géotechnique*, Vol. 41, No. 2, pp. 281-284, DOI: 10.1680/geot.1991.41.2.281.

Schmertmann, J. H. (1969). "Swell sensitivity." *Géotechnique*, Vol. 19, No. 4, pp. 530-533, DOI: 10.1680/geot.1969.19.4.530.

Shibuya, S., Jung, M., Chae, J., and Fujiwara, T. (2008). "Evaluation of short-term stability for sea-wall structure at kobe airport." *KSCE Journal of Civil Engineering*, Vol. 12, No. 3, pp. 155-163, DOI: 10.1007/s12205-008-0155-4.

Sun, D. A., Chen, B., and Wei, C. (2014). "Effect of fabric on mechanical behavior of marine clays." *Marine Georesources & Geotechnology*, Vol. 32, No. 1, pp. 1-17, DOI: 10.1080/1064119X.2012.710714.

Wang, Z. F., Shen, S. L., Ho, C. E., and Kim, Y. H. (2013). "Investigation of field installation effects of horizontal Twin-Jet grouting in Shanghai soft soil deposits." *Canadian Geotechnical Journal*, Vol. 50, No. 3, pp. 288-297, DOI: 10.1139/cgj-2012-0199.

Watabe, Y., Udaka, K., Kobayashi, M., Tabata, T., and Emura, T. (2008). "Effects of friction and thickness on long-term consolidation behavior of Osaka Bay clays." *Soils and Foundations*, Vol. 48, No. 4, pp. 547-561, DOI: 10.3208/sandf.48.547.

Wroth, C. P. and Wood, D. M. (1978). "The correlation of index properties with some basic engineering properties of soils." *Canadian Geotechnical Journal*, Vol. 15, No. 2, pp. 137-145, DOI: 10.1139/t78-014.

Yin, Z. Y., Chang, C. S., Hicher, P. Y., and Wang, J. H. (2011a). "Micromechanical analysis of the behavior of stiff clay." *Acta Mechanica Sinica*, Vol. 27, No. 6, pp. 1013-1022, DOI: 10.1007/s10409-011-0507-z.

Yin, Z. Y., Hattab, M., and Hicher, P. Y. (2011b). "Multiscale modeling of a sensitive marine clay." *International Journal for Numerical and Analytical Methods in Geomechanics*, Vol. 35, No. 15, pp. 1682-1702, DOI: 10.1002/nag.977.

Yin, Z. Y., Karstunen, M., Chang, C. S., Koskinen, M., and Lojander, M. (2011c). "Modeling time-dependent behavior of soft sensitive clay." *Journal of Geotechnical and Geoenvironmental Engineering*, *ASCE*, Vol. 137, No. 11, pp. 1103-1113, DOI: 10.1061/(ASCE)GT.1943-5606.0000527.

Yin, Z. Y., Yin, J. H., and Huang H. W. (2015). "Rate-dependent and long-term yield stress and strength of soft wenzhou marine clay: Experiments and modeling." *Marine Georesources & Geotechnology*, Vol. 33, No. 1, pp. 79-91, DOI: 10.1080/1064119X.2013.797060.

Yoon, G. L. and Kim, B. T. (2006). "Regression analysis of compression index for Kwangyang marine clay." *KSCE Journal of Civil Engineering*, Vol. 10, No. 6, pp. 415-418, DOI: 10.1007/BF02823980.

Zeng, L. L., Hong, Z. S., and Cui, Y. J. (2015). "Determining the virgin compression lines of remolded clays at different initial water contents." *Canadian Geotechnical Journal*, Vol. 52, No. 9, pp. 1408-1415, DOI: 10.1139/cgj-2014-0172.

Zeng, L. L., Hong, Z. S., Cai, Y. Q., and Han, J. (2011). "Change of hydraulic conductivity during compression of undisturbed and remolded clays." *Applied Clay Science*, Vol. 51, No. 1, pp. 86-93, DOI: 10.1016/j.clay.2010.11.005.

Zhu, Q. Y., Yin, Z. Y., Hicher, P. Y., and Shen, S. L. (2015). "Nonlinearity of one-dimensional creep characteristics of soft clays." *Acta Geotechnica*, pp. 1-14, DOI: 10.1007/s11440-015-0411-y.