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COMPRESSIBILITY CHARACTERISTICS OF CLAYS IN THE RED RIVER DELTA

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Abstract

The Red River Delta (RRD) is the second largest delta in Vietnam and is a significant economic zone in the country, encompassing important cities and economic zones. Geologically, the delta consists of clay layers distributed all over its area, which strongly impact the foundation designs of the infrastructure system (e.g., highways, industrial parks, harbors). However, up to date, there were no comprehensive studies on the compressibility characteristics of the clays in the delta. This study presents a primary study on some compressibility characteristics of clays in the whole delta with an emphasis on the use of CPTu data in the interpretation. For this, high-quality field and laboratory test results obtained from four research test sites and eight project sites were analyzed to depict the characteristics. The results indicate that the clays in the delta are typically soft to medium stiff and normally consolidated to slightly overconsolidated, which represent typical characteristics of young Holocene deposits. The compression index (C_c) of the clays was found to have good correlations with natural water content (W_n) or in-situ void ratio (e_0). The ratio of radial coefficient of consolidation from a consolidation test with a central drain (CD) to the vertical coefficient of consolidation from the standard consolidation test (i.e., $c_{r,CD}/c_v$ from the 4 research test sites is on average 2.76 (log t method) and 2.32 (root t method) and these average ratios are found typical for deltaic soil deposits. The correlations for the remolded samples indicate that the drainage type has a strong influence to the induced coefficients of homogeneously remolded soils. Results from this study help geotechnical engineers have a general view on the compressibility characteristics of the clays in the delta, which in turn helps the engineers secure optimal foundation design solutions.

Keywords: Red River delta; clays; CPTu; compression index; coefficients of consolidation.

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1. Introduction

The Red River Delta (RRD) is the second largest delta in Vietnam, and it plays a significant role in the economics development of the country in both agriculture and industrial sectors. The delta encompasses the capital city of Hanoi, neighbor provinces (e.g., Bac Ninh, Hai Duong, Hung Yen) with important industrial parks, and the port city of Hai Phong. In term of economics, the delta contributes a significant amount over the whole economy of the country. For example, the statistics in 2021 [1] indicated that, with a total area (15,080 km^2) of just 4.5% of the country's area (331,020 km^2), the GDP of the region was up to 26.8 % of that of the country and the contribution rate steadily increases

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further in the coming years. The significant development in economics recently is attributed to the development of the infrastructure system in the delta, which leads to the development of other economic sectors.

For infrastructure projects, such as highways, logistic facilities, harbors, industrial parks, optimal designs of foundation system are highly required since the foundation cost of these project types often accounts for large percent of the total project cost. In securing such optimal designs, properly understanding and using the compressibility characteristics of clayey soils in the designs are crucially important since the characteristics are driving factors for possible excessive deformation of the foundations. In the last two decades, mechanical characteristics of clays (e.g., compression index (C_c), coefficient of consolidation (c_v), and OCR value) at some places in Hai Phong city have been reported [2–4]. Geological profiles and physio-mechanical properties from boreholes along some sections of three national highways (NH) in the delta (NH No. 21, NH No. 38B, and NH No. 18) were also analyzed and depicted in Hien [5] and Hien and Giao [6]. The studies above provided us understanding on the characteristics of clays at some specific locations in the delta, however it is still not possible to draw a general picture of the characteristics in the delta, especially the connection between geological and geotechnical features of the delta.

This paper presents a study on some compressibility characteristics of clays in the delta with an emphasis on the use of CPTu data in the interpretation. For this, soil sampling for laboratory tests and CPTu were experimentally carried out at four research sites. In addition, laboratory and field test data at eight additional sites performed by FECON Corp were also used to form a database for the analyses.

2. Geological overview of the RRD

The Red River Delta (RRD) is located on the western coast of the Tonkin Gulf (Gulf of Bac Bo) with an area of about 13,030 km^2. The delta has a triangle shape with its height of about 150 km long from its apex (Viet Tri City at the Northwest) to the Southeast coastline, which is around 146 km from Quang Yen town to Day River mouth. Topographically, the RRD can be divided into three main zones, namely, upper delta, middle delta, and lower delta [7]. The upper delta lies at about 10−15 m above the mean sea level (MSL) and has slightly eroded paleo-terraces at the north side. The middle delta lies at an elevation of 5−6 m above the MSL with gentle wave-shaped in surface and good drainage condition. The lower delta lies near the shoreline with an elevation of 1−2 m above the MSL and composes of coastal flood plain or strand plain, saltwater marshes and sand bars.

Fig. 1 shows a quaternary geological and topographical map of the RRD and adjacent areas [8]. As shown, most of the delta area was formed in the sedimentation process throughout the Holocene period (9 kyr BP to date). Based on the dominated depositional environments, the sediments in the delta can broadly be divided into three main regions: fluvial-dominated, tide-dominated, and wave-dominated. In the north, north-west, west, south-west, the delta plain is surrounded by mountains which are composed of Precambrian crystalline rocks and Paleozoic to Mesozoic sedimentary rocks.

Using the core samples obtained from different locations within the delta, Tanabe et al. [8, 9] and Funabiki et al. [10], among several others, have analyzed and depicted significant findings for the sediments of the delta. Some key points from the studies might be summarized as follows: (1) due to the sea level rise from approximately 100 m below the present MSL (at 15 kyr BP) to the maximum level of about 2−3 m above the present MSL (from 6 kyr to 4 kyr BP), the Quaternary and Holocene sediments in the delta were strongly influenced by both fluvial and marine environments. Stratigraphic cross sections in the delta show the presence of fluvial sediments, estuarine sediments

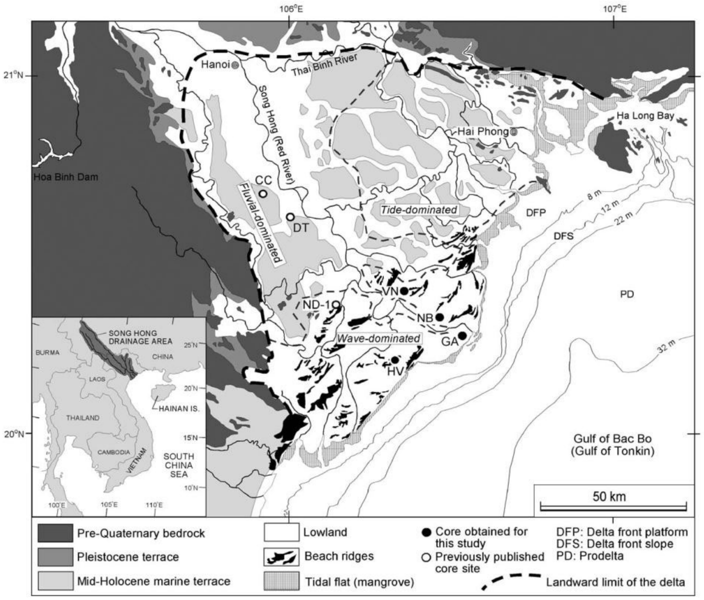


Figure 1. Quaternary geological and topographical map of RRD and adjacent areas [8]

and deltaic sediments [9]; (2) the sediment accumulation curves (attitude vs age) from Tanabe et al. [8] and Funabiki et al. [10] indicate that up to the depth of about 40 m (from the present MSL), all deposits found from the cores are Holocene sediments with the age of less than 10 kyr; (3) from the age of sediments obtained from radiocarbon dating (^{14}C), the progradation of the delta from 9 kyr to present was depicted [8]. The paleogeographic maps of the progradated delta are very helpful to geotechnical engineers to understand the behavior of the deposit regarding the age. It might be deduced from this summary that clay layers in the delta, the target of this study, are typically from the Holocene period.

3. Field and lab test programs

3.1. Study sites

To investigate the compressibility characteristics of the clays in the delta, especially the vertical and horizontal coefficients of consolidation, a research program of field and laboratory tests was carried out recently by the authors. For field tests of this research, soil sampling and cone penetration test with excess pore water pressure measurement (CPTu) were experimentally carried out at four test sites, namely: (1) Dinh Vu Industrial Zone (DVIZ), Hai An Dist., Hai Phong City; (2) Vietnam Singapore Industrial Park (VSIP), Thuy Nguyen Dist., Hai Phong City; (3) Kim Chung – Di Trach Urban Area (KC), Hoai Duc Dist., Hanoi; and (4) Nam Dinh Thermal Power Plant

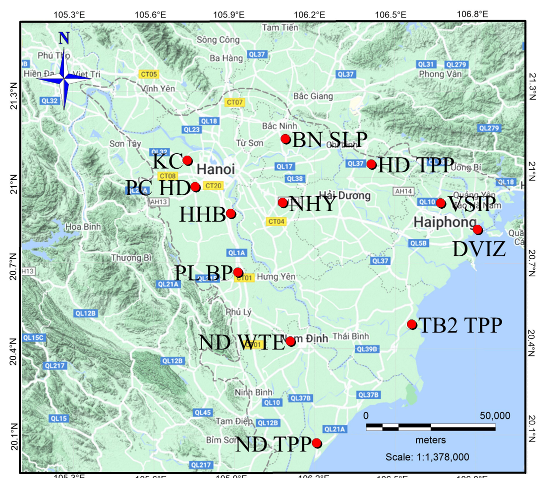


Figure 2. Location of the study sites

(ND TPP), Hai Hau Dist., Nam Dinh Province. Locations of the test sites are marked on the map shown in Fig. 2. Details of the test program are described in next section. Note that these research test sites were also the geotechnical investigation sites of industrial projects where other field tests were also extensively carried out.

Besides test results from the research program, test results from field and lab tests at eight test sites of remarkable industrial projects, which were conducted by FECON Corporation, were selected to make a database of boreholes and soil profiles. The test sites in this collection include: (1) Heineken Hanoi Brewery (HHB), Thuong Tin Dist., Hanoi; (2) Park City Ha Dong (PC HD), Ha Dong Dist., Hanoi; (3) Nestle Hung Yen (NHY), My Hao Dist., Hung Yen Province; (4) Hai Duong Thermal Power Plant (HD TPP), Kinh Mon Dist., Hai Duong Province; (5) Phu Ly Bypass (PL BP), Duy Tien Dist. & Kim Bang Dist., Ha Nam Province; (6) Thai Binh 2 Thermal Power Plant (TB2 TPP), Thai Thuy Dist., Thai Binh Province; (7) Bac Ninh SLP Park (BN SLP), Bac Ninh City, Bac Ninh Province; (8) Nam Dinh Greenity TWE Power Plant (ND WTE), My Loc Dist., Nam Dinh Province. Locations of the test sites are also shown in Fig. 2. The coordinates of all the test sites are given in Table 1.

Table 1. Coordinate of the study sites

No.	Site name	Longitude	Latitude	Province
1	DVIZ	106° 48’ 35.96"	20° 48’ 56.56"	Hai Phong
2	VSIP	106° 40’ 21.79"	20° 54’ 30.67"	Hai Phong
3	KC	105° 43’ 44"	21° 3’ 22.92"	Hanoi
4	ND TPP	106° 12’ 31.21"	20° 4’ 3.11"	Nam Dinh
5	HHB	105° 53’ 22.92"	20° 52’ 15.17"	Hanoi
6	PC HD	105° 45’ 27.86"	20° 57’ 48.38"	Hanoi
7	NHY	106° 5’ 0.96"	20° 54’ 31.5"	Hung Yen
8	HD TPP	106° 24’ 51.8"	21° 2’ 39.62"	Hai Duong
9	PL BP	105° 54’ 58.79"	20° 39’ 54.32"	Ha Nam
10	TB2 TPP	106° 33’ 50.33"	20° 29’ 0.92"	Thai Binh
11	ND WTE	106° 6’ 40.72"	20° 25’ 24.46"	Nam Dinh
12	BN SLP	106° 5’ 39.66"	21° 7’ 56.57"	Bac Ninh

3.2. *Field test program*

At each test site of the research program, soil boring and sampling were conducted to obtain undisturbed clay samples for laboratory tests, and the CPTu and dissipation test were conducted to study the CPTu-based soil stratification, classification and interpretations. The quantity (i.e., number of borehole and penetration holes) of the tests are listed in Table 2.

For soil boring and sampling, the borehole was advanced by using the rotary wash method with the use of bentonite slurry to stabilize the borehole walls. At a sampling depth, a thin-walled fixed-piston tube sampler of 1.0 m long and 76 mm in inner diameter was hydraulically pushed down to collect undisturbed clay samples. Right after the sample tube was retrieved at the ground surface, the tube ends were cleaned and filled with paraffin and then carefully sealed by tape to preserve water content as well as integrity of the soil sample. All the boring and sampling procedures were carried out in accordance with ASTM D1452–09 [11] and ASTM D1587–09 [12], respectively. When the sampling at the borehole was finished, the sample tubes were then carefully transported to the laboratory for lab tests.

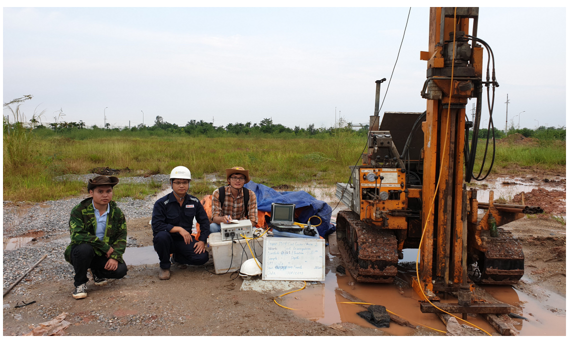
Table 2. Quantity of soil sampling and CPTu test at the four research test sites

No.	Test site	Boring & Sampling			CPTu		
		Number of boreholes	Sampling depths (m)	Number of tubes	Number of Penetration holes	Penetration depths (m)	Number of dissipation points
1	DVIZ	1	8.0–21.0	6	1	0–35	7
2	VSIP	2	6.5–17.5	10	1	0–17.0	5
3	KC	1	7.5–25.5	10	1	0–27.0	9
4	ND TPP	1	2.0–21.0	12	1	0–25.0	7

At each site, the CPTu was conducted nearby the sampling borehole (with a distance of less than 3.0 m). The distance between sampling borehole and penetration hole was strictly controlled to make sure that it is not too close to avoid possible soil disturbance and it is not far to ensure the compatibility of subsoil layers from the two holes. The CPTu was conducted following recommended procedures in ASTM D5778–12 [13]. For each penetration hole, a total of 5 to 6 dissipation test points were conducted to measure the excess pore water pressure (u_2) dissipated with time, which was then used to estimate the horizontal coefficient of consolidation of the clays. Fig. 3 shows moments of CPTu test at the four test sites.

(a) At DVIZ site

(b) At KC site



(c) At VSIP site

(d) At ND TPP site

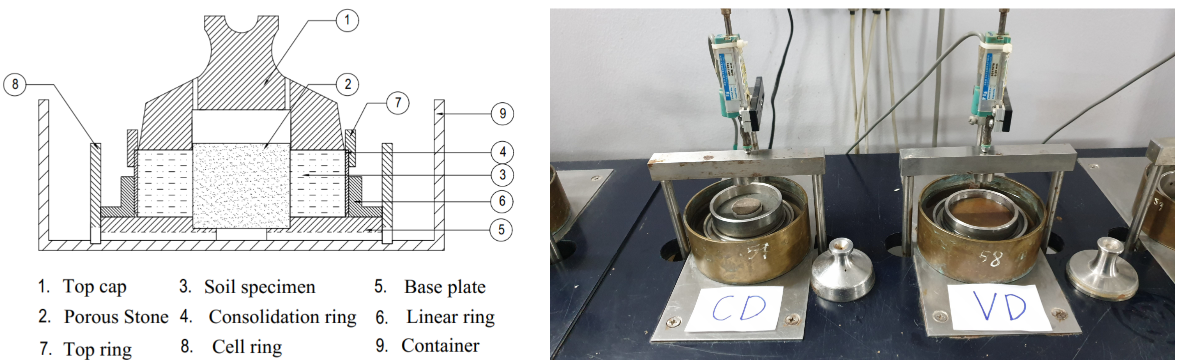
Figure 3. CPTu test at the four research test sites

Besides the field tests purposely conducted in the research program, other field tests, such as boring and sampling, standard penetration test (SPT), CPTu, Downhole seismic test, Field vane shear (FVS) test, Groundwater level monitoring, were also extensively carried out at the four research test sites as well as at the eight project sites. However, due to space limitation, those field tests are not described in detail herein. Together with the research test results, field and lab test results from the twelve test sites are valuable sources for the analyses in the paper.

3.3. Lab test program

At each test site, standard physical tests and the consolidation test using incremental loading method [14] were carried out on clay specimens to obtain basic physical properties (e.g., unit weight (γ), water content (W_n), particle size (d_p), specific gravity (G_s), liquid and plastic limits (LL and PL), and consolidation properties of clays at the site. In addition to these standard tests, other mechanical (shear) tests such as, unconfined compression (UC) test, unconsolidated undrained triaxial compression (UU) test, consolidated undrained compression (CU) test, and direct shear test (DS) were also carried out for most the sites. All the laboratory tests were carried out at geotechnical laboratory of FECON Corporation (Las XD 442).

In additional to the lab tests described above, pairs of consolidation test with radial drainage using a central drain (CD) and standard consolidation test with vertical drainage (VD) were carried out on clay specimens of the four research sites (i.e., DVIZ, VSIP, KC, ND TPP) to determine radial and vertical coefficient of consolidation (c_r and c_v), respectively. The CD test was performed by using the standard consolidation equipment (i.e., VD) system with the consolidation cell modified to house a central porous stone as graphically illustrated in Fig. 4(a). For the modified consolidation cell, basically, the bottom porous stone (of the standard cell) is replaced by a base steel plate (no. 5 in Fig. 4(a)) with a central hole connected to drainage slots on the bottom face, and the top cap (no. 1 in Fig. 4(a)) is cylindrically hollowed above the central porous stone. The central porous stone was vertically lowed and fit into a central hole of the specimen, which was created by using a mini thin-walled sampler having the same diameter to that of the central porous stone. The verticality and centricity of the central sampled hole was secured by a steel top cap (over the consolidation ring) with a central hole, working as a guide ring. In this study, the central porous stone was manufactured to have a diameter of 29.3 mm, which results in a ratio of diameter of soil specimen (d_e) over the diameter of the central porous stone (d_w) of 2.05 (i.e., $n = d_e/d_w = 2.05$).



(a) Cell design

(b) Use of the cell in the laboratory

Figure 4. Consolidation cell of the RCT with a central porous stone

In the laboratory, soil sample were extruded and cut into 10 cm pieces, which were all carefully wrapped in plastic membrane and then waxed to preserve water content of the samples. For a designated test depth, the 10 cm soil piece was cut into 3 equal parts, one for physical tests, one for the CD test and one for the VD test. It was therefore practically assumed that the soil specimens for the CD and VD had the same characteristics (e.g., homogeneity, compressibility, and permeability). For a typical test pair, seven loading steps of 12.5, 25.0, 50.0, 100.0, 200.0, 400.0, and 800.0 kPa were applied on the specimens. For each loading step, the test method B in ASTM D2435–11 [14] was applied to complete the loading step when 100% primary consolidation is reached. For this, the time – settlement curve of each loading step was observed carefully to make sure that the settlement slightly went beyond the point of 100% primary consolidation. Fig. 4(b) shows an example of the CD and VD tests in the laboratory and Fig. 5 shows an example of time – settlement curves obtained from the two tests at applied pressure σ_v = 800 kPa.

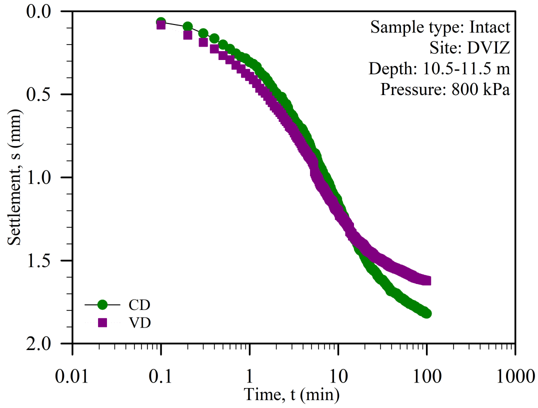


Figure 5. Example of time-settlement curves from CD and VD test

To examine the influence of heterogeneity and natural structures of soil to the consolidation characteristics, VD and CD tests were also carried on a number of remolded specimens. For this, soil specimens were completely remolded in the lab to make sure that the soil was homogeneously mixed and natural structures were broken. Table 3 shows a summary of quantity of CD and VD tests for the four test sites.

Table 3. Summary of CD and VD tests done for the four research test sites

Test site	Intact soil specimen		Remolded soil specimen	
	CD specimen (test curve)	VD specimen (test curve)	CD specimen (test curve)	VD specimen (test curve)
DVIZ	11 (66)	18 (108)	-	-
VSIP	32 (192)	32 (192)	3 (18)	-
KC	19 (114)	18 (108)	6 (36)	5 (30)
ND TPP	14 (84)	14 (84)	-	-
Total	76 (456)	82 (492)	9 (54)	5 (30)

4. Compressibility characteristics

4.1. CPTu-based stiffness

Stiffness (or consistency) of the clay is an important characteristic that needs to be described in every soil investigation report of sites having clay layers. The feature can be manually assessed in the field by a visual-manual procedure [15] or by the SPT N value (e.g., TCVN 9351:2012 [16], FHWA [17]). Although the CPTu is one of the most versatile field tests described in detail in many references (e.g., Mayne [18], Schnaid [19], Robertson and Cabal [20]), the stiffness classification of clayey soil based on CPTu data has been recommended in a few reports, e.g., Senneset et al. [21], Mayne and Kulhawy [22]. According to Senneset at et al. [21] and Mayne and Kullhawy [22], the stiffness of clayey soil is classified as follows: (*i*) Very soft: $q_t < 0.25$ (MPa); (*ii*) Soft: 0.25 (MPa) $\leq q_t < 0.5$ (MPa); (*iii*) Medium stiff: 0.5 (MPa) $\leq q_t < 1.0$ (MPa); (*iv*) Stiff: 1.0 (MPa) $\leq q_t < 2.0$ (MPa); and Very stiff: $q_t \geq 2.0$ (MPa). This classification is applied to the study sites herein.

Fig. 6 shows q_t profiles in clay layers from 10 sites in the database in which the boundaries of the stiffness ranges described above are given by vertical dashed lines. As shown, a common feature is that the q_t value (or the stiffness) increases rather linearly with depth at all the sites, indicating that the clays are rather homogeneous and were formed under relatively static depositional environments.

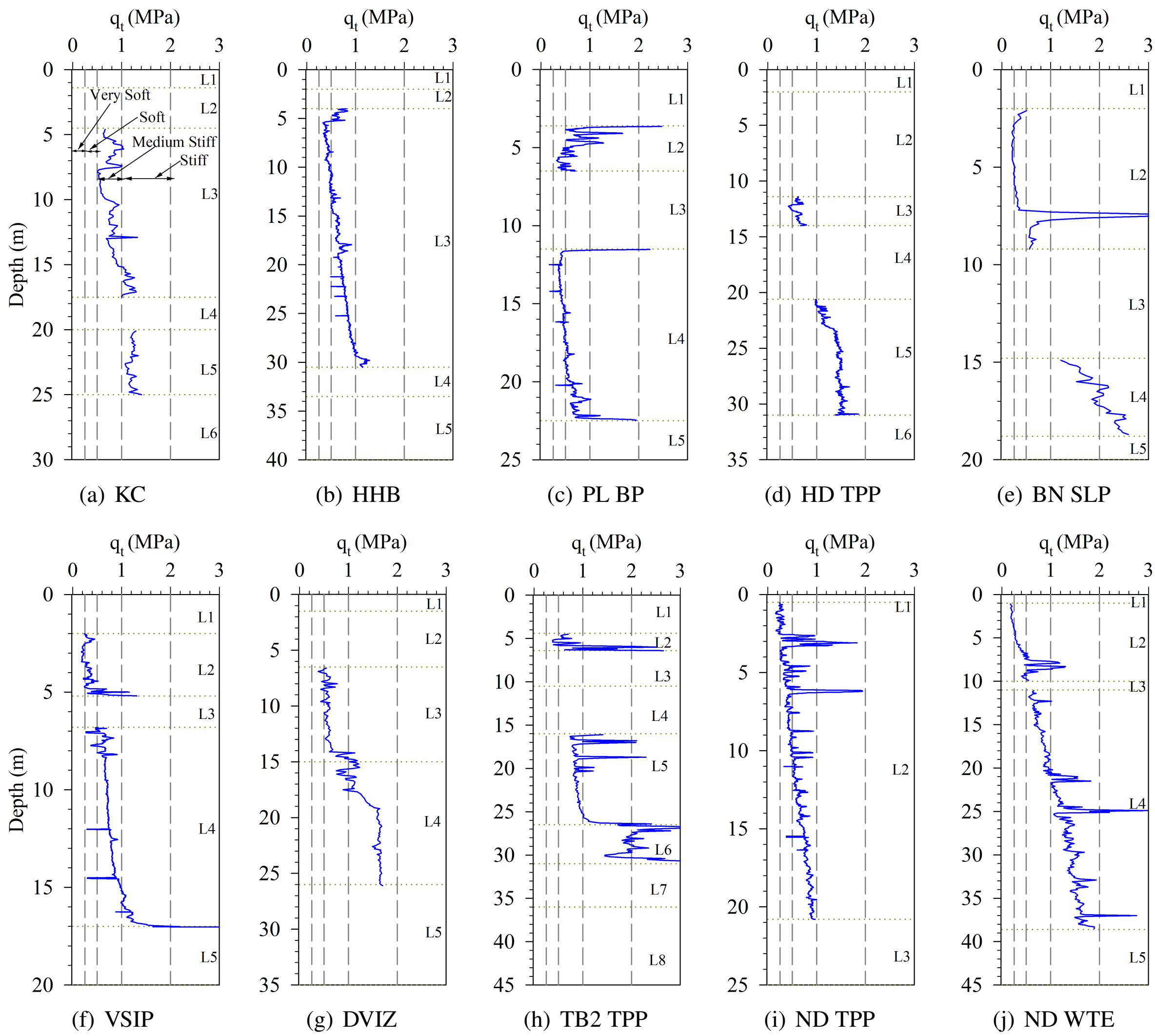


Figure 6. CPTu-based classification of stiffness for clayey soils at the ten test sites

The q_t profiles in Fig. 6 indicate that the clay layers at all depths at some sites (e.g., HHB, PL BP, VSIP, ND TPP) are soft to medium stiff ($0.5 < q_t < 1.0$ MPa) whereas the clay layers at the other sites are soft to medium in the upper depths and medium stiff to stiff in the lower depths, such as KC (20.0–25.0 m), HD TPP (21.0–31.0 m), BN SLP (15.0–20.0 m), DVIZ (19.0–26.0 m), TB 2 TPP (26.5–31.0 m), ND WTE (21.0–39.0 m). The variation of stiffness found in this study reflects typical characteristic of soil stiffness in the deltaic depositional environments: the deeper the soil is the stiffer it becomes. Besides, the stiffness of the soil in the delta can also be inferred from the age of deposits. The accumulation curves (age – depth plots) of soils in the delta [8] show that up to the depth of 20 m the age of the soil deposits is mostly less than 5 kyr (young deposits), and it is found in this study that the clays in such depths are soft and medium stiff. From depth of 20.0 m to 40.0 m, the age of the soils at some places is up to 11 kyr, and logically soils are stiffer as some clay layers found medium stiff to stiff in this study.

4.2. Compression index (C_c)

The compression index (C_c) obtained from the oedometer consolidation test [14] is a key parameter to assess the compressibility of the clayey soil. Based on C_c value, the compressibility of the clayey soil may be classified as follows [23]: (*i*) Slight or low compressibility: $C_c < 0.20$; (*ii*) Moderate or intermediate: $0.2 \leq C_c < 0.4$; and (*iii*) High compressibility: $C_c \geq 0.4$.

The C_c value is typically determined from the $e - \log \sigma'_v$ curve of the test. Besides, the index can also be evaluated by the following equation [24].

$$C_c = (\ln 10)(1 + e_0)\frac{\sigma'_v}{M} \tag{1}$$

where e_0 is the initial void ratio of the soil specimen; σ'_v is an effective stress value in the range from which C_c is graphically determined; M is the constrained modulus at σ'_v.

In this study, the C_c value obtained from the $e - \log \sigma'_v$ curve was compared with that obtained from Eq. (1), for which $\sigma'_v = \sigma'_{v0}$ (the in-situ effective stress at the depth of soil specimen) and the $M = M_0$ (at σ'_{v0}) were simply applied. The M_0 value was obtained from correlations with q_t recommended in Lune et al. [24] as follows: (1) $M_0 = 2q_t$ (for $q_t < 2.5$ MPa) and (2) $M_0 = 4q_t - 5$ (for 2.5 MPa $< q_t <$ 5 MPa).

Fig. 7 shows the variation of C_c values with depth at the same sites (Fig. 6) obtained from the $e - \log \sigma'_v$ and from Eq. (1), in which the vertical dashed lines depict the boundaries of the stiffness ranges described above. It is interesting to note that the C_c values obtained from the $e - \log \sigma'_v$ curve and from Eq. (1) resulted in rather similar trend and values with depth. The figure indicates that most clay layers at the sites are in moderate to high compressibility ($C_c \geq 0.2$) except the clay layers at HD TPP and BN SLP sites. It is interesting that the low compressibility indicated by C_c at HD TPP and BN SLP sites also agree with the stiffness resulted from the q_t profiles shown in Fig. 6. In fact, these two sites are located near the northeast border of the delta, where the clay has been deposited early in the propagation of the delta (Fig. 10 in Tanabe et al. [8]).

Note that the q_t value in Fig. 6 steadily increases rather linearly with depth, indicating that the stiffness of the clays also increases with depth, i.e., the C_c value should theoretically be decreased steadily with depth. However, the C_c values obtained from both the methods in Fig. 7 expose to be relatively constant with depths, except the trend at HD TPP and TB2 TPP. In fact, if a homogeneous clay layer has constant e_0, the $C_c$value by Eq. (1) should be constant with depth (as $\sigma'_v/M = 1/m$ is a constant, where m = modulus number, a constant for a given soil layer), whereas the q_t value should increase linearly due to the influence the total stress level (σ_{v0}) at the test depth. Thus the relatively constant of trend of C_c in Fig. 7 may be attributed to the insignificant variation of e_0 with depth (as the clay layers herein are rather homogeneous), the stress relieve of the soil specimens in the laboratory (i.e. all specimens have $\sigma_{v0} = \sigma_{h0} = 0$ at the beginning of the test) and other possible effects.

The C_c value obtained from all the twelve sites were correlated with the natural water content (W_n) and the in-situ void ratio (e_0). To have more representative correlations, some outliers of C_c/W_n and

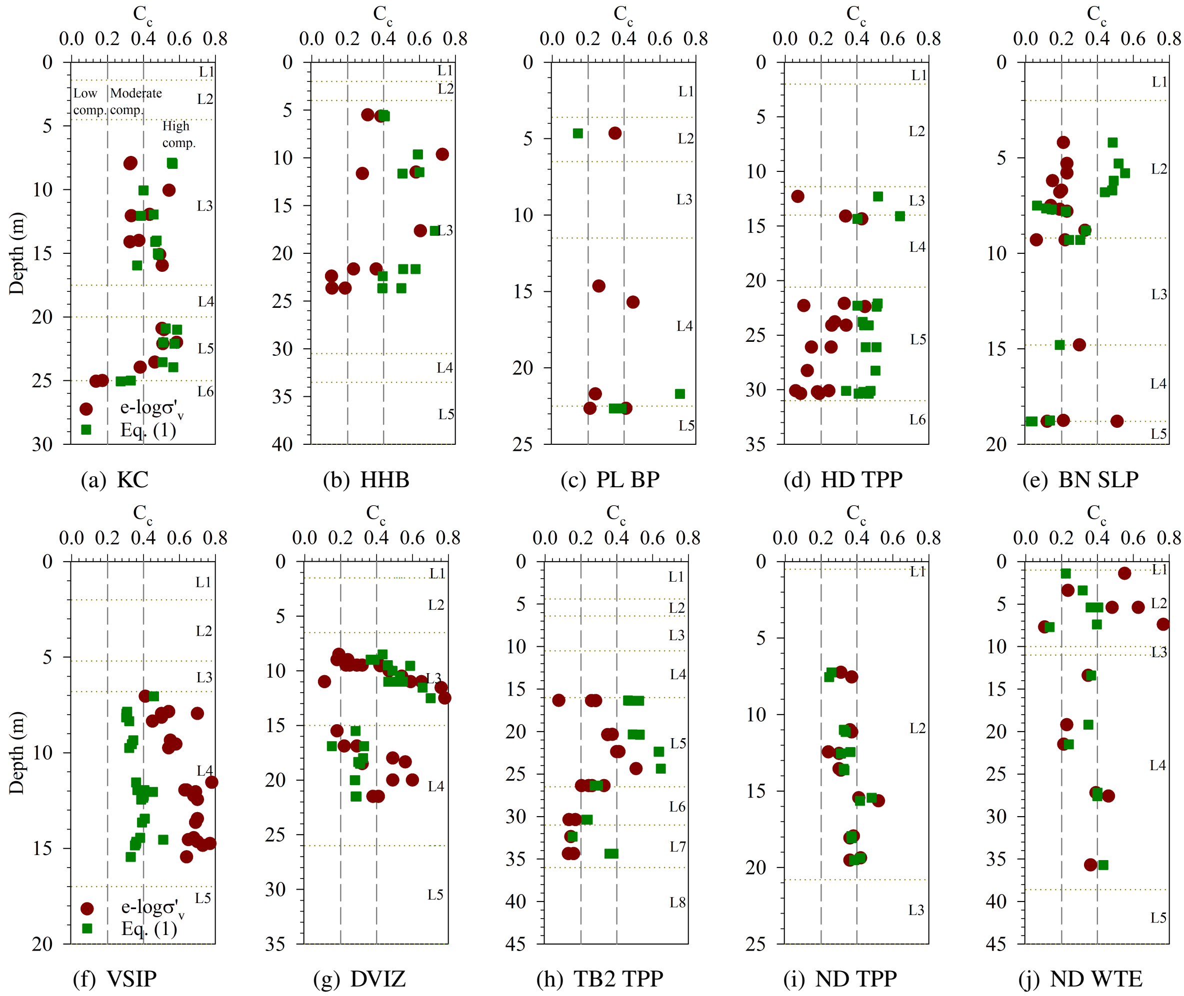


Figure 7. Variation of C_c with depth at the ten test sites

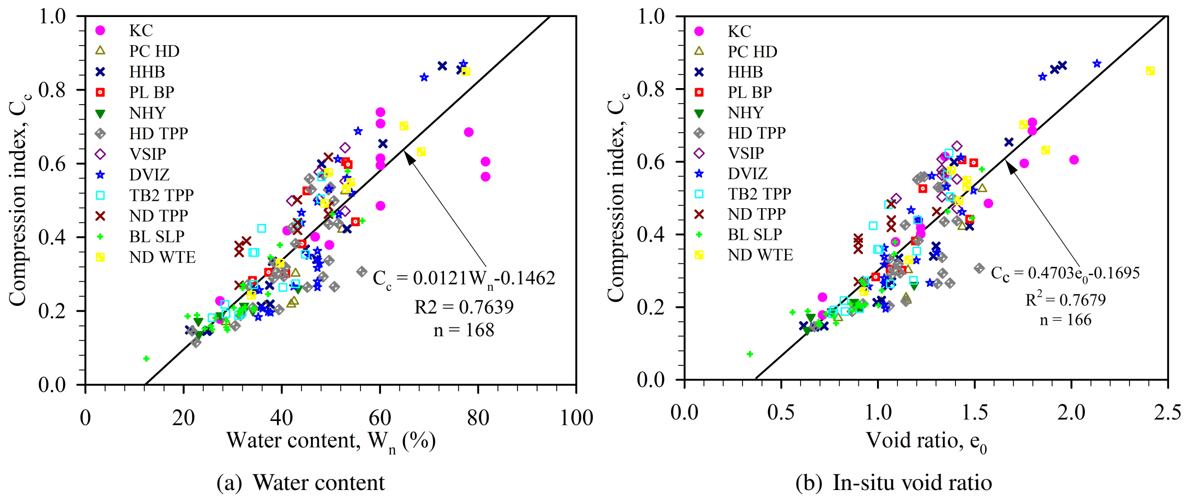


Figure 8. Correlations between C_c and W_n , C_c and e_0

C_c/e_0 ratios were removed to retain approximately of 68% of the data population under the normal distribution curve [25].That is, the ratios out of the range of $(\mu - \sigma) < C_c/W_n$ (or C_c/e_0) $< (\mu + \sigma)$ were removed, where μ and σ are the mean and standard deviation of corresponding ratio. Fig. 8 shows the correlations after such outliers were removed. Note that the correlations shown in Fig. 8 are rather similar to ones summarized in Tables 6.10 and 6.11 in an FHWA report [26] for different clays in the world.

4.3. Overconsolidation ratio (OCR)

The preconsolidation stress (σ'_p) and overconsolidation ratio ($OCR = \sigma'_p/\sigma'_{v0}$) are important compressibility characteristics of clayey soil in estimating settlement of foundations and other behavioral features. In this study, the σ'_p of the clay at the sites were estimated from the $e - \log \sigma'_v$ curve obtained from the 1D consolidation test and from the CPTu data as well.

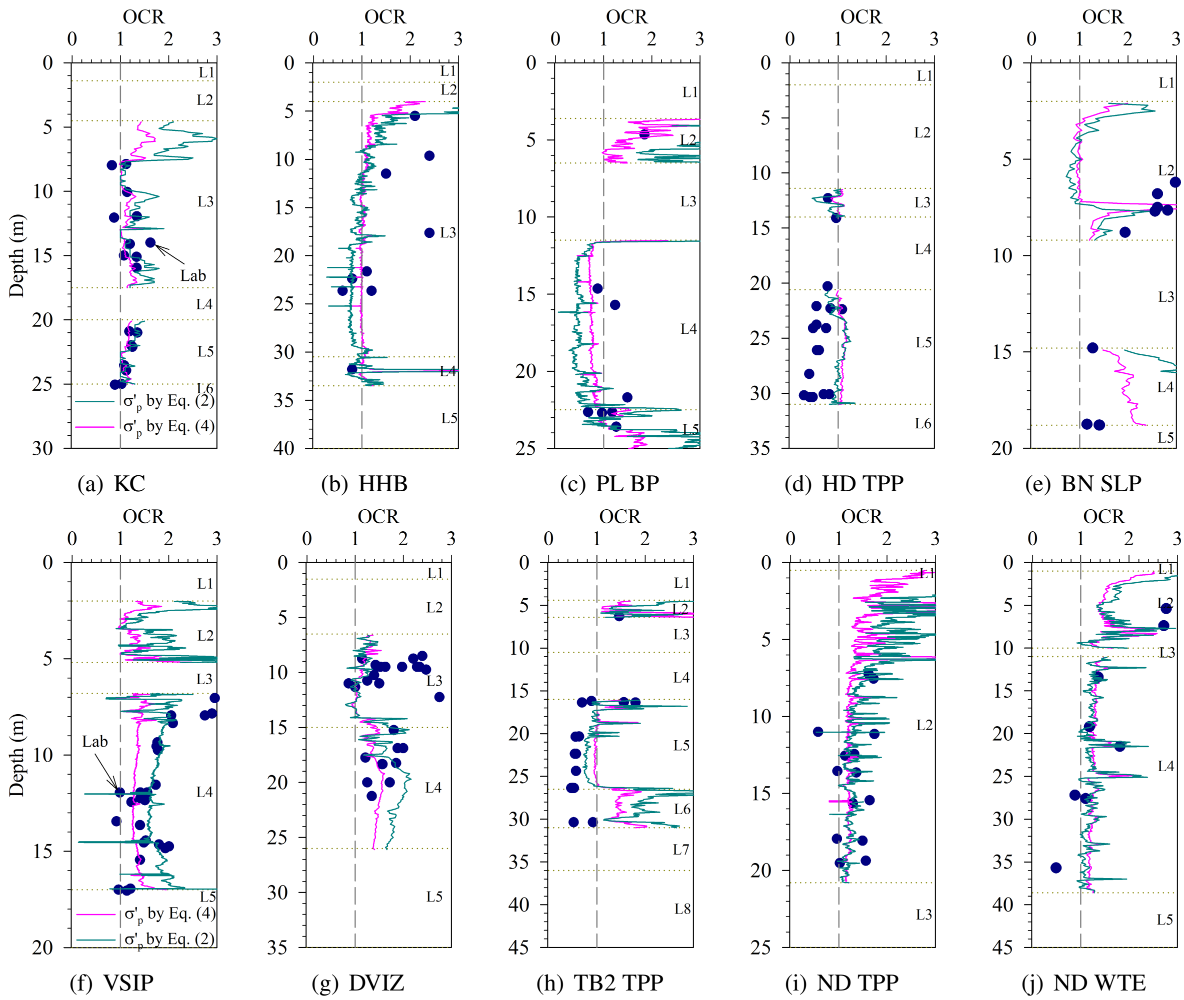


Figure 9. Variation of *OCR* with depth at the ten test sites

From an $e - \log \sigma'_v$ curve, there have been a number of methods proposed to estimate the σ'_p value, which are summarized in some references (e.g., Grozic et al. [27], Dung and Giao [28], Boon [29]). Although Casagrande method [30] has been a traditional one and recommended in many manuals, handbooks and standards (e.g., ASTM D2435-11 [14], CFEM [31], Day [32], FHWA [17]), the method is reported to be operator-dependent and influenced by scale effect [27, 33]. To avoid such effects and to deal with $e - \log \sigma'_v$ curves showing no distinct curvature around the σ'_p value (for which Casagrande method [30] is difficult to apply), the methods of Silva [34] and Becker et al. [35] are often recommended [33, 36]. In this study both Silva [34] and Becker et al. [35] methods were applied to estimate the σ'_p value of the test specimens from the twelve sites and both methods resulted in similar σ'_p values. Fig. 9 shows the variation of OCR with depth at the 10 sites (Figs. 6 and 7), in which the σ'_p value from $e - \log \sigma'_v$ curve was obtained from Becker et al. [35] method.

The σ'_p values at the ten sites were also estimated using CPTu data and compared with that obtained from the $e - \log \sigma'_v$ curve. For this, two CPTu-based recommended equations were applied to estimate the values, of which the first equation is [37]:

$$\sigma'_p = 0.33\,(q_t - \sigma_{v0})^{m'}\,(p_a/100)^{1-m'} \tag{2}$$

where $p_a = 101.3$ kPa is the atmospheric pressure, m' is an exponent related to soil type.

$$m' = 1 - \frac{0.28}{1 + (I_c/2.65)^{25}} \tag{3}$$

where I_c is soil behavior type index given in detail in [37].

The second equation is for mixed soil types [18, 37]:

$$\sigma'_p = 101\,(p_a)^{0.102}\,(G_0)^{0.478}\left(\sigma'_{v0}\right)^{0.420} \tag{4}$$

where G_0 is the maximum shear modulus of soil (in kPa). In this study, $G_0 = \rho V_s^2$ was applied and the shear wave velocity (V_s, in m/s) of clay was estimated from q_t (in kPa) as follows [18]:

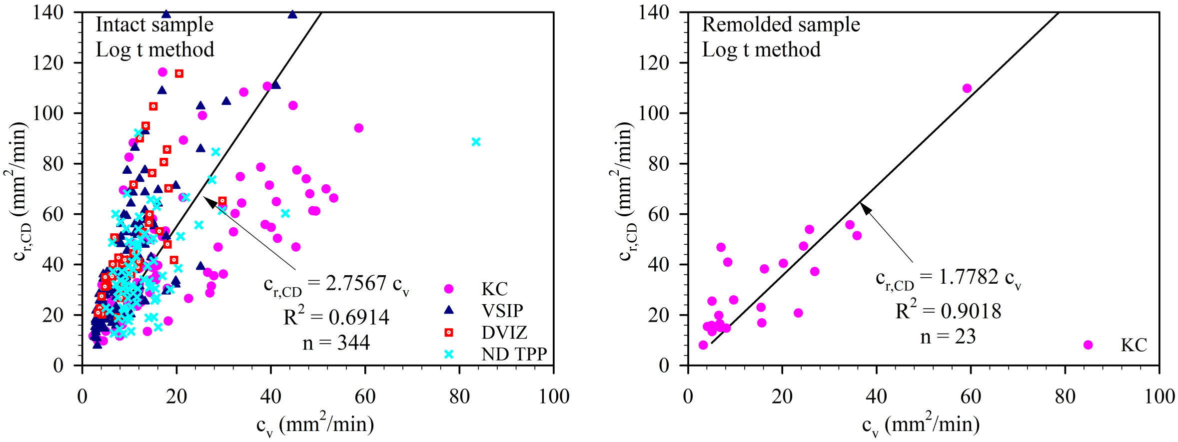
$$V_s = 1.75\,(q_t)^{0.627} \tag{5}$$

The OCR values, in which σ'_p was obtained from Eqs. (2) and (4) are also plotted in Fig. 9 for the comparison purpose. It is interesting to note two key features from the Fig. 9: (1) generally, the OCR values obtained from the $e - \log \sigma'_v$ curve agree well with the values obtained from CPTu-based methods; (2) OCR values from both the approaches indicate that the clay layer in the delta are typically normally consolidated ($OCR \approx 1$, but < 1.5) to slightly overconsolidated ($OCR = 1.5$–4) [38].

Examining closely at the OCR profiles one may notice that some OCR values from both consolidation test and CPTu data at some depths are less than 1.0. This doesn't mean that the soil at these depths is underconsolidated. In fact, the $OCR < 1$ from the consolidation test (e.g., at HD TPP and TB2 TPP) may come from several reasons such as the heterogeneity (e.g., clay consists of thin lenses of sand or some shell fragments) and unavoidable soil disturbance effect, which led to flatter time – settlement curves. The effect of heterogeneity is clearly indicated in the analysis of coefficient of consolidation in the next section. On the other hand, the $OCR < 1$ from the CPTu data (e.g., at HHB and PL BP sites) may be attributed to the exception of some clays that do not fit to the database used to develop the correlations (i.e., Eqs. (2) and (4)).

4.4. *Coefficients of consolidation (c_v and c_r)*

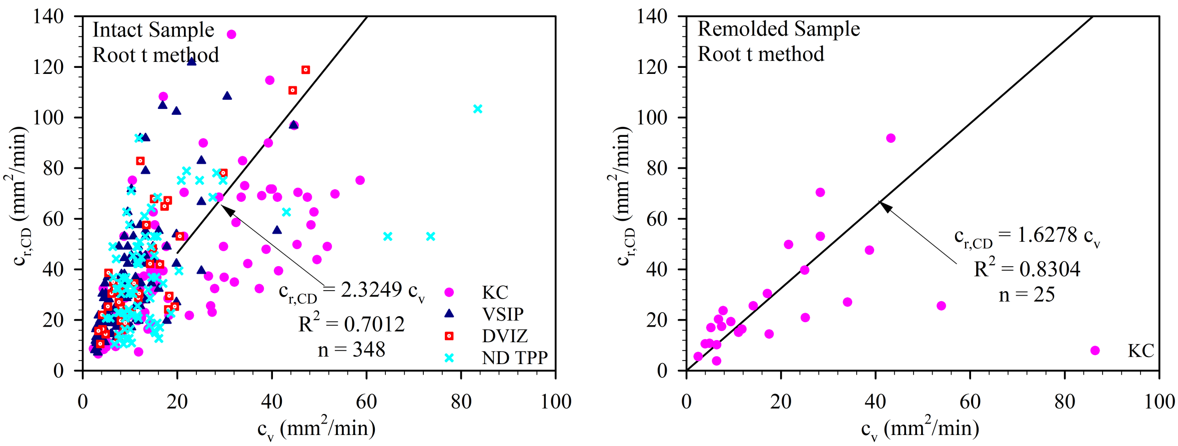
In the literature, there are a number of methods proposed to estimate the vertical coefficient of consolidation (c_v) and radial coefficient of consolidation ($c_{r,CD}$) from the time – settlement curves (Fig. 5). For a VD test, the log t and root t methods have been commonly recommended in handbooks and standards (e.g., ASTM D2435-09 [14]) and they are adopted herein. For a CD test, the log t method [39] and root t method [40, 41] were also adopted for comparable analyses. In fact, log t method (or root t method) for vertical and radial drainage types (i.e., VD and CD tests) uses the same features of the time – settlement curves to derive c_v and $c_{r,CD}$, thus the coefficients from the same method are in principle comparable for the two drainage types. Due to space limitation, detailed procedures of the methods are not described herein.



(a) Intact sample (b) Remolded sample

Figure 10. Comparison of $c_{r,CD}$ and c_v for log t method

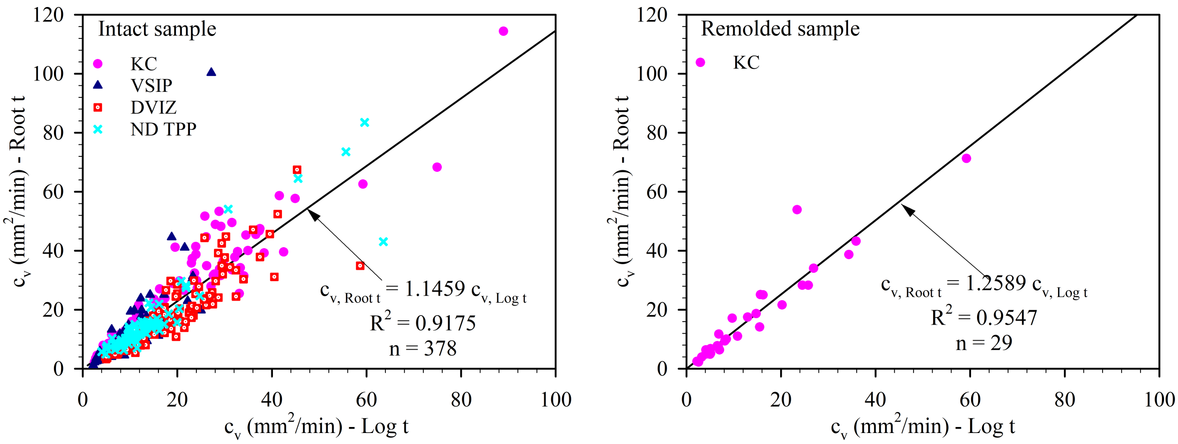
Following the procedures of the log t and root t methods for VD and CD tests, $c_{r,CD}$ and c_v values were obtained from the time – settlement curves of the test pairs (Table 3) and correlations were developed for the coefficients. Figs. 10, 11, 12, 13 show correlations of the coefficients after outliers, which lie out of the range of $(\mu - \sigma)$ < variable < $(\mu + \sigma)$, were removed as similarly done for the correlations of C_c value shown in Fig. 8.



(a) Intact sample (b) Remolded sample

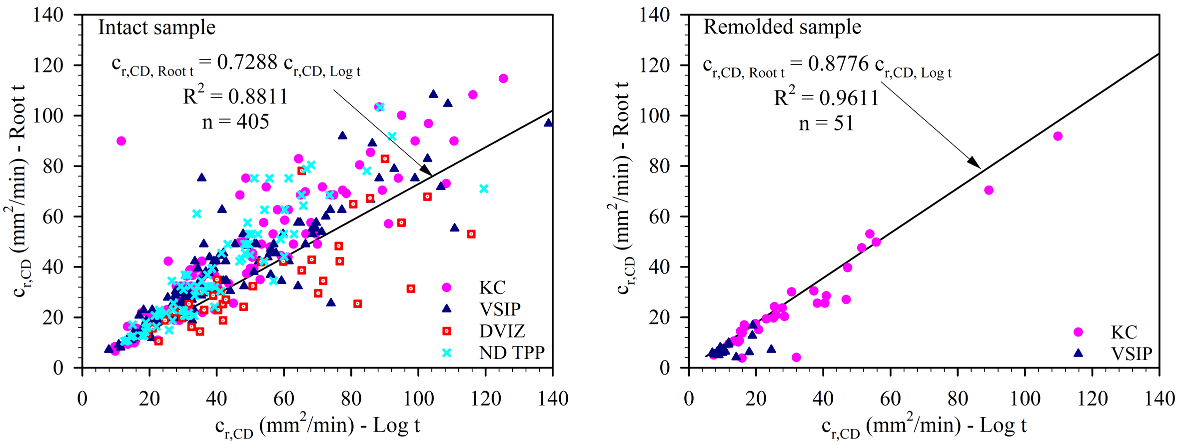
Figure 11. Comparison of $c_{r,CD}$ and c_v for root t method

Figs. 10(a) and 11(a) indicate that although the outliers were removed the data points still distribute quite scatteredly, indicating the strong influence of the heterogeneity of intact soil samples. The correlations in the figures show that on average the ratios of $c_{r,CD}/c_v$ are 2.75 and 2.32 for log t and root t methods, respectively, and these ratios are typically found for many natural soils. For the remolded samples (Figs. 10(b) and 11(b)), one might expect that $c_{r,CD}/c_v$ would be around one since the soil was remolded to be homogeneous. However, it is interesting to note from the figures that on average the $c_{r,CD}/c_v$ ratios are 1.78 and 1.63 for log t and root t methods, respectively, which are larger than one. The ratio of larger than one indicates that, besides the structure of natural soil, the drainage type and drainage length also strongly influence to the induced coefficient of consolidation.



(a) Intact sample (b) Remolded sample

Figure 12. Comparison of c_v from root t and log t methods



(a) Intact sample (b) Remolded sample

Figure 13. Comparison of $c_{r,CD}$ from root t and log t methods

It is interesting to judge the correlations of the same parameter obtained from different methods. In such an attempt, Fig. 12(a) and 12(b) show correlations of c_v obtained from log t and the value obtained from root t method for intact samples and remolded sample, respectively. Similarly, Figs. 13(a) and 13(b) show the correlations of $c_{r,CD}$ value from the two methods.

Figs. 12(a) and 12(b) indicate that c_v obtained from root t method for the intact and remolded samples is on average 1.15 times and 1.26 times, respectively, the value obtained from log t method. The findings herein agree well with common recommendations that c_v obtained from root t method is slightly larger than that obtained from log t method [42, 43]. Inversely, Figs. 13(a) and 13(b) show that $c_{r,CD}$ obtained from root t method for the intact and remolded samples is on average 0.73 times and 0.87 times, respectively, the value obtained from log t method. The inverse of the $c_{r,CD,Root\ t}/c_{r,CD,Log\ t}$ ratio, which is consistent for intact and remolded samples, implies that not the structure and intactness of soil but the drainage type is the key factor resulting in different ratio of the coefficients from the two methods. Table 4 shows a summary of coefficients obtained from Figs. 12 and 13.

Table 4. Coefficients of correlations shown in Figs. 12 and 13

No.	Correlations	Intact samples			Remolded samples		
		α	R^2	n	α	R^2	n
1	c_v vs $c_{r,CD}$ (Log t)	2.7567	0.6914	344	1.7782	0.9018	23
2	c_v vs $c_{r,CD}$ (Root t)	2.3249	0.7012	348	1.6278	0.8304	25
3	c_v (Root t) vs c_v (Log t)	1.1459	0.9175	378	1.2589	0.9547	29
4	$c_{r,CD}$ (Root t) vs $c_{r,CD}$ (Log t)	0.7288	0.8811	405	0.8776	0.9611	51

5. Conclusions

A database of methodical and well-performed field and laboratory test results from four research test sites and eight project sites over the RRD were analyzed to depict some compressibility characteristics of clays in the delta. The following are key conclusions drawn from this study: (*i*) the corrected cone resistance (q_t) and compression index (C_c) profiles have indicated that clays in the delta are typically soft to medium stiff in the upper depths and medium stiff to stiff in the lower depths at some places. The stiffness increase rather linearly with depth, indicating that the clays are rather homogeneous and were formed relatively static depositional environments; (*ii*) the C_c value of the clays was found to have good correlations with natural water content (W_n) and in-situ void ratio (e_0) and the correlations are similar to ones recommended in manuals and reports; (*iii*) the preconsolidation stress (σ'_p) values obtained from the standard consolidation test as well as from CPTu data (and therefore the overconsolidation ratio, *OCR*) have indicated that the clays in the delta are typically normally consolidated (NC) to slightly overconsolidated with *OCR* mostly from 1.0 to 2.0; (*iv*) the ratio of radial coefficient of consolidation (central drain) to the vertical coefficient of consolidation (i.e., $c_{r,CD}/c_v$) from the 4 research sites is on average 2.76 (log t method) and 2.32 (root t method) and these average ratios are found typical for deltaic soil deposits. For the remolded samples, this ratio is about 1.7 (not 1.0 as one might expect), indicating that the drainage type has strong influence on the induced coefficients.

The findings in this study were obtained from limited test sites in the delta. Since the delta is very large, more test sites with valuable field and lab test data are expected to characterize the compressibility characteristics of the delta more comprehensively.

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